0007B	Failure of track owner to use qualified persons to inspect track.
0007B2	Failure of track owner to have persons demonstrate required knowledge, ability to detect deviations and prescribe remedial action, restoration and renewal.
0007B3	Failure of person to have written authorization for inspection.
0007C	Failure of track owner to use qualified persons to inspect, restore or renew CWR.
0007C2	Failure to complete comprehensive CWR training course.
0007C3	Failure of track owner to have persons demonstrate required knowledge, ability to detect deviations, CWR.
0007C4	Failure of person to have written authorization to inspect, restore or renew CWR.
0007D	Failure of track owner to use not fully qualified persons to pass trains over broken rails or pull aparts.
0007D2	Train speed exceeds 10 mph over broken rails or pull aparts.
0007D3	Person not watching or prepared to stop train movements over broken rails or pull aparts.
0007D4	Failure to promptly notify and dispatch person(s) fully qualified under 213.7 to the location of the broken rail or pull apart.
0007E	Failure of track owner to properly maintain written records of designation and basis for each designation.
0009B1	Failure to restore other than excepted track to compliance with class 1 stds. Within 30 days after a person designated under 213.7(a) has determined that operations may safely continue over defect(s) not meeting class 1 or excepted track standards.
0009B2	Failure of track owner to enforce, over class 1 defects, the limiting conditions imposed by person designated under 213.7(a).
0011	Proper qualified supervision not provided at work site during work hours when track is being restored or renewed under traffic conditions.
0013	Failure to add dynamic movement to static measurement
0033A1	Drainage or water-carrying facility not maintained.

0033A2	Drainage or water-carrying facility obstructed by debris.
0033A3	Drainage or water-carrying facility collapsed.
0033A4	Drainage or water-carrying facility obstructed by vegetation.
0033A5	Drainage or water-carrying facility obstructed by silting.
0033A6	Drainage or water-carrying facility deteriorated to allow subgrade saturation.
0033A7	Uncontrolled water undercutting track structure or embankment.
0037A	Combustible vegetation around track-carrying structures.
0037B1	Vegetation obstructs visibility of railroad signs and fixed signals.
0037B2	Vegetation obstructs visibility of grade crossing warning signs and signals by the traveling public.
0037C1	Vegetation interferes with railroad employees performing normal trackside duties.
0037C2	Vegetation obstructs passing of day and night signals by railroad employees.
0037C3	Excessive vegetation in toepaths and around switches that interferes with employees performing normal trackside duties.
0037D	Vegetation prevents proper functioning of signal and/or communication lines.
0037E1	Excessive vegetation at train order office, depot, interlocking plant, a carman's building, etc., prevents employees on duty from visually inspecting moving equipment when their duties so require.
0037E2	Excessive vegetation at train meeting points prevents proper inspection by railroad employees of moving equipment.
0037E3	Vegetation brushing sides of rolling stock that prevents employees from visually inspecting moving equipment from their normal duty stations.
0053A	Gage measurement improper
0053B1	Gage dimension on tangent track exceeds allowable.

0053B2	Gage dimension on tangent track.is less than allowable
0053B3	Gage dimension on curved track exceeds allowable
0053B4	Gage dimension on curved track.is less than allowable
0053B5	Gage dimension for excepted track.exceeds allowable
0055A1	Alinement deviation of tangent track for a 62-foot chord exceeds allowable
0055A2	Alinement deviation of curved track for a 62-foot chord exceeds allowable.
0055A3	Alinement deviation of curved in class 3-5 track for a 31-foot chord exceeds allowable.
0057A1	Maximum crosslevel on a curve in class 1 and 2 track exceeds allowable.
0057A2	Maximum crosslevel on curve in class 3-5 track exceeds allowable.
0057B1	Operating speed exceeds allowable for 3-inches of unbalance, based on curvature and elevation.
0057C1	Operating speed exceeds allowable for 4-inches of unbalance, based on curvature and elevation.
0057D	Operating speed exceeds allowable for a fra approved unbalance based on curvature and elevation for contiguous high speed track exceeds allowable.
0059A	Where fixed physical conditions are not considered, operating speed based on curvature and actual minimum elevation in a curve exceeds allowable
0059B	Improper elevation runoff in a spiral exceeds allowable
0063A1	Runoff in any 31-feet of rail at end of raise exceeds allowable.
0063A10	Crosslevel differences in all of six or more consecutive pairs of staggered joints in class 2-5 track exceeds allowable.
0063A2	Deviation from uniform profile on either rail exceeds allowable.
0063A3	Deviation from zero crosslevel at any point on tangent track exceeds allowable

0063A4	Reverse crosslevel on curve track exceeds allowable
0063A5	Difference in crosslevel (warp) between any two points less than 62-feet apart on tangent track exceeds allowable.
0063A6	Difference in crosslevel (warp) between any two points less than 62-feet apart on curve track between spirals exceeds allowable.
0063A7	Difference in crosslevel (warp) between any two points less than 62-feet apart on spiral track exceeds allowable.
0063A8	Variation in crosslevel per 31-feet on a physically restricted length spiral exceeds allowable.
0063A9	Where elevation at any point in curve track equals or exceeds six inches, the difference in crosslevel within 62-feet between that point and a point with greater elevation exceeds allowable
0103A	Fouled or insufficient ballast failing to transmit and distribute loading
0103B	Fouled or insufficient ballast failing to restrain the track laterally, longitudinally or vertically.
0103C	Fouled ballast failing to provide adequate drainage for the track.
0103D	Fouled or insufficient ballast failing to maintain proper geometry.
0109A	Crossties made of unsound material
0109B1i	39-foot segment of track does not have sufficient number of crossties to provide effective support to hold gage within limits prescribed in part 213.53(b).
0109B1ii	39-foot segment of track does not have sufficient number of crossties to provide effective support that will maintain surface within the limits prescribed by part 213.63.
0109B1iii	39-foot segment of track does not have sufficient number of crossties to provide effective support that will maintain alinement within the limits prescribed by part 213.55.
0109B2	Crossties not effectively distributed to support a 39-foot segment of track.
0109B3	No effective support ties within the prescribed distance from a joint.

0109B4	Failure to maintain the minimum number of crossties per fra track class for each 39-foot segment of track as indicated in table in this section.
0109C1	Crossties, other than concrete, that meet the minimum requirements of part 213.109 (b)(4),are broken through.
0109C2	Crossties, other than concrete, that meet the minimum requirements of part 213.109 (b)(4), are split or otherwise impaired to the extent the crosstie will allow the ballast to work through, or it will not hold spikes or rail fasteners.
0109C3	Crossties, other than concrete, that meet the minimum requirements of part 213.109 (b)(4), are so deteriorated that the crosstie plate or base of the rail can move laterally 1/2-inch relative to the crosstie.
0109C4	Crossties, other than concrete, that meet the minimum requirements of part 213.109 (b)(4), are cut by the crosstie plate through more than 40 percent of a crosstie's thickness.
0110A	Failure to maintain and operate GRMS within minimum design requirements over designated GRMS line segments
0110A1	Failure to notify fra at least 30 days prior to the designation of a GRMS line segment
0110A2	Failure to notify fra at least 10 days prior to the removal of a line segment from GRMS designation
0110B1	Failure to provide required information identifying a GRMS line segment
0110C	Failure to provide sufficient technical data to establish compliance with minimum GRMS design requirements
0110G	Failure of GRMS to provide analog trace of specified parameters
0110H	Failure of GRMS to provide exception report listing of specified parameters
0110I	Failure to provide exception report listing to par. 213.7 individual prior to next inspection required under par. 213.333
0110J1i	Failure to maintain and make available documented calibration procedures on GRMS vehicle
0110J1ii	Failure to initiate a daily instrument verification procedure
0110J2	Failure to maintain PTLF accuracy within five-percent of 4,000 reading
0110K	Failure to meet training requirements.

0110L	Failure to initiate required remedial action for exceptions listed on GRMS record of lateral restraint
0110M1i	Gage widening exceeds allowable measured with PTLF
0110M5	Failure to provide functional PTLF to par. 213.7 individual whose territory is subject to requirements of par. 213.110
0110M6	Failure to restore contact between rail and lateral rail restraint components
0110N	Failure to keep GRMS records as required
0110O	Failure to conduct GRMS inspections at required frequency
0113A	Operation continued over defective rail without required remedial action.
0113B	Rail defect originating from bond wire attachment [where a defect results from a bond wire attachment, fra inspectors must cite this defect code and also include a description of the applicable rail defect as described in §213.113]
0113B1	Transverse fissure
0113B10	Ordinary break
0113B11	Damaged rail
0113B12	Flattened rail
0113B13	Bolt-hole crack
0113B14	Broken or defective weld
0113B15	Head web separation
0113B2	Compound fissure
0113B3	Horizontal split head
0113B4	Vertical split head

0113B5	Split web
0113B6	Piped rail
0113B7	Broken base
0113B8	Detail fracture
0113B9	Engine burn fracture
0115A1	Rail-end mismatch on tread of rail exceeds allowable.
0115A2	Rail-end mismatch on tread of rail exceeds allowable (CWR).
0115A3	Rail-end mismatch on gage side of rail exceeds allowable.
0115A4	Rail-end mismatch on gage side of rail exceeds allowable (CWR).
0118A1	Failure of track owner to develop and implement written CWR procedures.
0118A2	Failure to comply with written CWR procedures.
0118A3	Failure of track owner to develop a training program for the implementation of their written CWR procedures.
0118C	Failure of track owner to comply with existing CWR plan.
0118E1	Failure of track owner to file a revised CWR plan with associate administrator of safety/chief operating officer within 30 days of revision.
0118E2	Failure of track owner to re-submit a conforming plan within 30 days of receipt of final submission decision.
0119A	Failure to comply with written CWR procedures - installation and adjustment
0119B	Failure to comply with written CWR procedures - anchoring or fastening requirements
0119C	Failure to comply with written CWR procedures - joint installation and maintenance procedures
0119D	Failure to comply with written CWR procedures - maintaining desired rail installation temperature range

0119E	Failure to comply with written CWR procedures - curved track
0119F	Failure to comply with written CWR procedures - train speed
0119G	Failure to comply with written CWR procedures - physical track inspections
0119H	Failure to comply with written CWR procedures - CWR joint inspection
0119I	Failure to comply with written CWR procedures - training
0119J	Failure to comply with written CWR procedures - recordkeeping
0119K	Car procedures and revisions not available at job site or maintained in one manual
0121A1	Rail joint not of structurally sound design and dimension (jointed track)
0121A2	Rail joint not of structurally sound design and dimension (CWR).
0121B1	Cracked or broken joint bar in classes 3 through 5 track (other than center-break) (jointed track)
0121B2	Cracked or broken joint bar in classes 3 through 5 track (other than centerbreak) (CWR)
0121B3	Cracked or broken insulated joint bar in classes 3 through 5 track (other than centerbreak) (CWR).
0121B4	Worn joint bar allows excessive vertical movement of rail in joint in classes 3 through 5 track (jointed track)
0121B5	Worn joint bar allows excessive vertical movement of rail in joint in classes 3 through 5 track (CWR).
0121C1	Center cracked or broken joint bar (jointed track)
0121C2	Center cracked or broken joint bar (cwr).
0121C3	Center cracked or broken insulated joint bar (cwr)
0121D1	Less than 2 bolts per rail at each joint for conventional jointed rail in classes 2 through 5 track.
0121D2	Less than 1 bolt per rail at each joint for conventional jointed rail in class 1 track.
0121E	Less than 2 bolts per rail at any joint in continuous welded rail.

0121F1	Loose joint bars (jointed track)
0121F2	Loose joint bars (cwr).
0121G1	Torch-cut or burned-bolt hole in rail in classes 2 through 5 track (jointed track)
0121G2	Torch-cut or burned-bolt hole in rail in classes 2 through 5 track (cwr).
0121H1	Joint bar reconfigured by torch cutting in classes 3 through 5 track (jointed track)
0121H2	Joint bar reconfigured by torch cutting in classes 3 through 5 track (cwr).
0122Ai	Torch cut rail applied in class 3 through 5 track for other than emergency.
0122Aii	Failure to remove torch cut rails within specified time frame.
0122B1	Failure to remove non-inventoried torch cut rail within 30 days of discovery.
0122B2	Train speed exceeds allowable over non-inventoried torch cut rail.
0123A	Insufficient tie plates in class 3 through 5 track.
0123B	Object between base of rail and the bearing surface of the tie plate causing concentrated load.
0127A	Failure of fastening components to effectively maintain gage within the limits described in part 213.53(b)
0127A2	Insufficient fasteners in a track segment.
0127A3	Insufficient fasteners at rail joint.
0127B	Failure of applied rail anchors to provide effective longitudinal restraint
0127C	Failure of fastener placement at insulated joints from performing as intended, or the crosstie does not effectively support the rail
0133A1	Loose, worn, or missing switch clips.
0133A10	Missing switch, frog, or guard rail plates.

0133A11	Loose or missing switch point stops.
0133A12	Loose, worn, or missing frog bolts.
0133A13	Loose, worn, or missing guard rail bolts.
0133A14	Loose, worn or missing guard rail clamps, wedge, separator block, end block, or other components.
0133A15	Turnout or track crossing fastenings not intact or maintained.
0133A16	Obstruction between switch point and stock rail.
0133A17	Obstruction in flangeway of frog.
0133A18	Obstruction in flangeway of guard rail.
0133A2	Loose, worn, or missing clip bolts (transit, side jaw, eccentric, vertical).
0133A3	Loose, worn, or defective connecting rod.
0133A4	Loose, worn, or defective connecting rod fastening.
0133A5	Loose, worn, or defective switch rod.
0133A6	Loose, worn, or missing switch rod bolts.
0133A7	Worn or missing cotter pins.
0133A8	Loose or missing rigid rail braces.
0133A9	Loose or missing adjustable rail braces.
0133B	Insufficient anchorage to restrain rail movement.
0133C	Flangeway less than 1 1/2 inches wide.
0135A1	Stock rail not securely seated in switch plates.
0135A2	Stock rail canted by overtightening rail braces.

0135B1	Improper fit between switch point and stock rail.
0135B2	Excessive lateral or vertical movement of switch point.
0135B3	Lateral or vertical movement of a stock rail adversely affecting the fit of the switch point to the stock rail.
0135C	Outer edge of wheel contacting gage side of stock rail.
0135D	Heel of switch insecure.
0135E1	Switch stand or switch machine insecure or operable with excessive lost motion.
0135E2	Connecting rod insecure or operable with excessive lost motion.
0135F	Throw lever operable with switch lock or keeper in place.
0135G	Switch position indicator not clearly visible.
0135H1	Unusually chipped or worn switch point.
0135H2	Improper switch closure due to metal flow.
0135I	Use of tongue and plain mate where speeds exceed class one.
0137A	Insufficient flangeway depth.
0137B	Frog point chipped, broken, or worn in excess of allowable.
0137C	Tread portion of frog worn in excess of allowable.
0137D	Use of flange bearing frog where speed exceeds that permitted by class 1.
0137E	Severe frog condition not otherwise provided. (advisory only cannot be used solely to recommend violation)
0139A	Outer edge of wheel contacting side of spring wing rail.
0139B	Toe of wing rail not fully bolted and tight.
0139B1	Ties under or wing rail not solidly tamped.

0139C1	Bolt-hole defect in spring frog.
0139C2	Head and web separation in spring frog.
0139D	Insufficient compression in spring to hold wing rail against point rail.
0139E	Excessive clearance between hold-down housing and horn.
0141A	Raised guard worn excessively.
0141B	Frog point rebuilt before restoring guarding face.
0143A1	Guard check gage less than allowable.
0143A2	Guard face gage exceeds allowable.
0143A3	Cracked or broken guard rail.
0205A	Derail not clearly visible.
0205B	Derail operable when locked.
0205C1	Loose, worn, or defective parts of derail.
0205C2	Insecure derail or stand
0205D1	Improper size derail.
0205D2	Improperly installed derail.
0233A	Track inspected by other than qualified designated individual.
0233B	Track being inspected at excessive speed.
0233B1	One inspector inspecting more than two tracks or inspecting tracks with centers greater than allowable.
0233B2	Two inspectors inspecting more than four tracks or inspecting tracks with centers greater than allowable.
0233B3i	Main track not traversed within the required frequency.

0233B3ii	Siding track not traversed within the required frequency.
0233C	Failure to inspect at required frequency.
0233D	Failure to initiate remedial action for deviations found.
0234B1	Failure to inspect at required frequency on class 4 and 5 main track and class 3 main track with regularly scheduled passenger service, exceeding 40 million gross tons annually, at least twice each calendar year, with no less than 160 days between inspect
0234B2	Failure to inspect at required frequency on class 4 and 5 main track and class 3 main track with regularly scheduled passenger service equal to or less than 40 million gross tons annually, at least once per calendar year.
0234B3	Failure to inspect at required frequency on class 3, 4, and 5 main track with exclusively passenger service, either an automated inspection or walking inspection once per calendar year.
0234B4	Failure to inspect at required frequency in accordance with paragraph (b)(1) or (b)(2) of this section because of train operation interruption.
0234C	Sections of tangent track greater than 600 feet constructed of concrete crossties, not inspected
0234D1	Automated inspection measurement system incapable of measuring and processing rail seat deterioration.
0234E	Failure of automated inspection measurement system to produce an exception report.
0234F	Failure to maintain and make available to fra a record of the inspection data and exception record for the track inspected.
0234G	Failure to maintain proper procedures for data integrity.
0234H	Failure to provide annual rail seat deterioration training.
0235A1	Failure to inspect turnouts at required frequency.
0235A2	Failure to inspect track crossings at required frequency.
0235A3	Failure to inspect lift rail assemblies or other transition devices on moveable bridges at required frequency.

0235B	Failure to operate specified switches in classes 3 through 5.
0235C	Switch, turnout, track crossing or transition device used less than once a month and not inspected on foot before use
0235C1	Track used less than once a month not inspected on foot before use
0237A	Failure to inspect rail for internal defects at required frequency.
0237B	Failure of equipment to inspect rail at joints.
0237C	Defective rail not marked properly.
0237E	Improper action taken after expiration limits of previous internal rail defect search.
0239A	Failure to conduct special inspections when required.
0241A	Failure to keep records as required.
0241B1	Failure of inspector to complete report the day of the inspection.
0241B2	Failure of inspector to sign report.
0241B3	Failure to indicate the nature of deviation.
0241B4	Failure of inspector to provide required information.
0241B5	Failure to record required periodic or follow-up cwr joint inspection
0241C	Failure of rail inspection record to provide required information.
0241D	Failure to make records available for copying and inspection.
0241E1	Electronic system does not maintain the integrity of each record.
0241E2	Electronic storage not initiated within 24 hours.
0241E3	Electronic system allows record or amendments to be modified.

0241E4	Electronic amendments not stored separately from record.
0241E4i	Person making electronic amendment not identified.
0241E5	Electronic system corrupts or losses data.
0241E6	Paper copies of records not made available for inspection and copying.
0241E7	Inspection reports not available to inspector or subsequent inspectors.

Appendix C – Rail Mill Branding and Key Dimensions

Weight	Type	Rail Mill/Branding Designations														
		U.S. Steel	Bethlehem	Illinois	Old Illinois	Carnegie	Tennessee	Lackawanna	Midvale	Colorado	Inland	Cambria	MD/PA	Dominion	Algoma	Sydney
70	ARA-A			7020		7020	7020	7031								
70	ARA-B		174	7030		7030	7030	7032								
70	ASCE	7040	70AS	7040	7010	7040	7040	700	532	701			237			
70	Bangor Aroostook		70-BA					703					97			
70	Chicago & Alton				7002											
70	Pennsylvania			7033	7005	7033	7033		504				57			
72	CP Sandberg															
72	Chicago NW	7250	72NP	7250	7201	7250	7250		581							
72	Spokane							722								
74	MD/PA												146			
75	ASCE	7540	75AS	7540	7506	7540	7540	750	529	753			214			
75	Boston & Maine		92					752					92			
75	Lackawanna		75-C					753								
75	Int. Great Northern			7551		7551	7551									
75	Miscellaneous															
75	Missouri Pacific	7550	75MP	7550	7512	7550	7550	754	528							
75	Nat. Ry. Mexico		128													
75	NYC. (Dudley)															
75	MD/PA												87			
75	Seaboard (Dudley)		75DY	7522		7522	7522						221			
75	Union Pacific		75-B	7523	7513	7523	7523			754			249			
75	Union Pacific	7524	75SP	7524		7524	7524			757						
76	MD/PA												216			
78	Great Northern				77501			775								
78	Old Colony		78-OC										98			
79	MD/PA												76			
80	Frictionless		79.5-C													
80	ARA-A	8020	80-RA	8020		8020	8020	8031		801						
80	ARA-B	8030	80-RB	8030		8030	8030	8032	569	802						
80	ASCE	8040	80AS	8040		8040	8040	800	530	800	8040		251			
80	Canadian Northern		804	8010		8010	8010	804								
80	DUDLEY	8022	80DY	8022		8022	8022						220			

Weight	Type	Rail Mill/Branding Designations																
		U.S. Steel	Bethlehem	Illinois	Old Illinois	Carnegie	Tennessee	Lackawanna	Midvale	Colorado	Inland	Cambria	MD/PA	Dominion	Algoma	Sydney
80	Frictionless		80-MC-F													
80	Great Northern				8009			802								
80	Hocking Valley								540							
80	New York Central		220	8022	8008	8022	8022	801	543							
85	Asce	8540	85AS	8540	8504	8540	8540	850	531	851	8540		235			
85	C.B. & Q.	8543	85-CB	8543	8506	8543	8543	855		852						
85	Canadian Pacific	8524	85CP	8524		8524	8524	856			8524			8501	113	
85	Head Free – CP													8504	137	
85	Denver & RG									850						
85	D. & R.G. / C & S									853						
85	Great Northern		854	8553	8509	8553	8553	854								
85	Missouri Pacific	8550	853	8550	8507	8550	8550									
85	N.Y.C. & Stl. / Kcs		85-NK	8521		8521	8521	8531			8521					
85	Pennsylvania	8531	85PS	8531	8530	8531	8531	8530	559		8531					
85	Pennsylvania		85-PR	8533	8503	8533	8533	852	500				67			
85	Seaboard (Dudley)		85DY	8522		8522	8522	851					261			
85	Soo Line	8520		8520		8520	8520									
85	Western Pacific															
90	ARA-A	9020	90RA	9020		9020	9020	9031	563	902	9020					
90	ARA-B	9030	90RB	9030		9030	9030	9032	561	905	9030					
90	ASCE	9040	90AS	9040	9002	9040	9040	900	535		9040		245			
90	A.T. & SF	9021	90SF	9021		9021	9021	9033		903	9021					
90	Chicago NW	9035	90OM	9035		9035	9035	904								
90	Denver Rio Grande									906						
90	Frictionless			9039		9039	9039									
90	Frictionless			9029		9029	9029									
90	Great Northern	9024	90GH	9024		9024	9024			908	9024					
90	Great Northern			9036		9036										
90	Great Northern		90-GN	9034	9010	9034	9034	9030	560	904						
90	Head Free - R.A.	9027	90RA-T	9027		9027	9027			TC1013						
90	Interborough R. T.	9050	90RT	9050		9050	9050	902					77			
90	Lehigh Valley															
90	N.Y.C. (Dudley)		90DY					901								
90	Union Pacific	9023		9023	9003	9023	9023			901						

Weight	Type	Rail Mill/Branding Designations														
		U.S. Steel	Bethlehem	Illinois	Old Illinois	Carnegie	Tennessee	Lackawanna	Midvale	Colorado	Inland	Cambria	MD/PA	Dominion	Algoma	Sydney
91	Lackawanna		91- DL	9133		9133	9133	911								
92	Frictionless		304													
93	Frictionless		93-NH-F					932								
95	ASCE							950					267			
95	Boston & Albany															
95	W & H Ry. (Dudley)		95-DY					951								
97	Frictionless		97-CO-F													
98	Frictionless		98-PS-F													
100	ARA-A	10020	100RA	10020		10020	10020	10031	565	1003	10020					
100	ARA-B	10030	100RB	10030		10030	10030	10032	564	1002	10030					
100	AREA	10025	100RE	10025		10025	10025			10025	10025					
100	ASCED	10040	100 AS	10040	10001	10040	10040	1000	536				247			
100	Canadian Pacific															100CP-RE
100	Chicago NW	10035	100-OM	10035		10035	10035	1006			10035					
100	Elgin Joliet & East.			10050		10050	10050									
100	Great Northern	10036	100GN	10036		10036	10036	1008								
100	Head Free - R.A.		100RA-T											10004	136	
100	Head Free - R.E.		100RE-T													
100	Interborough R. T.	10005	100RT	10005		10005	10005	1005								
100	N.Y., N.H. & H.	10034	100NH	10034	10004	10034	10034	1002					100			
100	New York Central		100-DY	10022	10003	10022	10022	1001								
100	Pennsylvania	10031	100PS	10031		10031	10031	10030	558		10031					
100	Pennsylvania	10033	100PR	10033	10002	10033	10033	1003	520				96			
100	Reading	10032	100RG	10032		10032	10032	1007								
100	R.W. Hunt.															
101	Lackawanna	10133	101DL	10133		10133	10133	10130								
105	Lackawanna	10533	105DL	10533		10533	10533	1052								
105	Dudley	10524	105DY	10524		10524	10524				10524					
105	New York Central		105-B	10522		10522	10522	1051								
106	Miscellaneous.									1060						
107	N.Y., N.H. & H.	10734	107NH	10734		10734	10734	1072								
110	AREA	11025	110RE	11025		11025	11025			1100	11025					
110	ASCE												268			
110	C.T.A.	11050														

Weight	Type	Rail Mill/Branding Designations														
		U.S. Steel	Bethlehem	Illinois	Old Illinois	Carnegie	Tennessee	Lackawanna	Midvale	Colorado	Inland	Cambria	MD/PA	Dominion	Algoma	Sydney
110	Great Northern	11036	110GN	11036		11036	11036				11036					
110	Head Free - AREA	11027	110RE-T	11027		11027	11027				11027					
110	Lehigh Valley	11033	110LV	11033		11033	11033									
112	AREA	11228	112RE	11228		11228	11228			1121	11228					
112	Head Free – R.E.	11227	112RE-T	11227		11227	11227				11227HF					
112	CB & Q – TR	11229		11229		11229	11229			1122						
113	Head Free – SP	11327	113RE-T	11327		11327	11327			1130						
115	AREA	11525	115RE	11525		11525	11525			1150	11525					
115	D.R.G.W.									1155						
115	Dudley	11522/23	115DY	11523		11523	11523									
115	Miscellaneous.															
118	Lackawanna		118DL-M													
119	Area	11937								1190	11937					
120	Area			12025		12025	12025									
120	Mfg. Std.		120-MS													
120	New York Central		120-DY					1201								
122	CB (B&O)		122-CB													
125	Pennsylvania		308	12531		12531	12531	12530	584							
126	Frictionless		125.5-PSF													
127	Dudley	12723	127DYM								12723					
127	New York Central		127-DY	12722		12722	12722				12722					
129	CB & Q – TR	12929		12929		12929	12929				12929					
130	AREA	13025	130RE	13025		13025	13025			1300	13025					
130	Head Free – P.S.		130PS-T													
130	Head Free – R.E.	13027	130RE-T	13027		13027	13027				13027			13001	138	
130	Phil. & Reading		130RG													
130	Pennsylvania	13031	130PS	13031		13031	13031	13030	589	1302	13031					
131	Area	13128	131RE	13128		13128	13128			1311	13128					
131	Head Free – R.E.	13127														
132	Area	13225	132RE	13225		13225	13225			1321	13225					
132	Head Free – S.P.	13227	132RE-T	13227		13227	13227			1320						
133	Area	13331	133RE	13331		13331	13331			1330	13331					
135	Central of NJ		135CR													
136	AREA	13637	136RE							1360						

Weight	Type	Rail Mill/Branding Designations														
		U.S. Steel	Bethlehem	Illinois	Old Illinois	Carnegie	Tennessee	Lackawanna	Midvale	Colorado	Inland	Cambria	MD/PA	Dominion	Algoma	Sydney
136	Lehigh Valley	13633	136LV	13633		13633	13633									
136	Lehigh Valley		136-LV													
136	Lehigh Valley		136-LV-M													
136	New York Central		136NYC													
140	AREA/PS	14031	140RE	14031		14031	14031									
141	AREA		141RE													
152	Pennsylvania	15222	152PS	15224		15224	15224									
155	Pennsylvania	15531	155PS	15531		15531	15531									

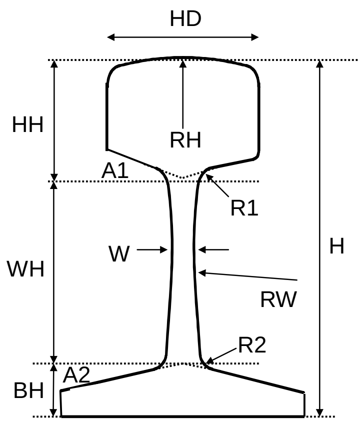
		Key Rail Dimensions											
		Rail Height	Head Width	Web Thickness	Head Height	Web Height Fishing	Base Height	Head Radius	Web Radius	Top Fillet Radius	Bottom Fillet Radius	Head Bottom Angle	Base Angle
Weight	Type	H	HD	W	HH	WH	BH	RH	RW	R1	R2	A1	A2
70	ARA-A	4 ¾	2 3/8	1/2	1 11/32	2 1/2	29/32	14	14	0.375	0.375	1 to 4	1 to 4
70	ARA-B	4 35/64	2 3/8	33/64	1 23/64	2 17/64	59/64	12	12	0.3125	0.3125	13 deg.	13 deg.
70	ASCE	4 5/8	2 7/16	33/64	1 11/32	2 15/32	13/16	12	12	0.25	0.25	13 deg.	13 deg.
70	Bangor Aroostook	4 ¾	2 7/16	1/2	1 13/32	2 19/32	3/4	12	12	0.25	0.25	12 deg.	12 deg.
70	Chicago & Alton	4 3/8	2 23/64	35/64	1 45/64	1 59/64	3/4					12 deg.	12 deg.
70	Pennsylvania	4 ½	2 7/16	1/2	1 19/32	2 1/8	25/32	10	8	0.25	0.25	13 deg.	13 deg.
72	CP (Sandberg)	4 15/16	2 1/4	1/2	1 5/8	2 25/64	59/64	6	VERT.	0.375	0.375	15 deg.	15 deg.
72	Chicago NW	4 ¾	2 3/8	9/16	1 13/32	2 1/2	27/32					14 deg.	14 deg.
72	Spokane Int'l. Ry.	4 45/64	2 7/16	33/64	1 27/64	2 15/32	13/16	12	12	0.25	0.25	13 deg.	13 deg.
74	MD/PA	4 11/16	2 7/16	9/16	1 3/4	2 3/16	3/4	15	15	0.3125	0.3125	17 deg.	13 deg.
75	ASCE	4 13/16	2 15/32	17/32	1 27/64	2 35/64	27/32	12	12	0.25	0.25	13 deg.	13 deg.
75	Boston & Maine	5	2 1/2	9/16	1 7/16	2 47/64	53/64	12	STR.	0.25	0.25	13 deg.	13 deg.
75	Lackawanna	4 11/16	2 1/2	1/2	1 43/64	2 13/64	13/16	10.5	10	0.3125	0.3125	18 deg.	12° 45'
75	Int. & Grt. Nor	4 ¾	2 1/2	9/16	1 7/16	2 15/32	27/32					13 deg.	13 deg.
75	Miscellaneous	4 ¾	2 1/2	1/2	1 27/32	2 1/8	25/32					13 deg.	13 deg.
75	Missouri Pacific	4 ¾	2 9/16	9/16	1 7/16	2 15/32	27/32	12	30	0.25	0.25	13 deg.	13 deg.
75	Nat. Ry. Mex.	5	2 3/4	1/2	1 3/8	2 7/8	3/4					12 deg.	12 deg.
75	N.Y.C. (Dudley)	5	2 5/8	17/32	1 3/8	2 3/4	7/8	14	14	0.5	0.3125	14 deg.	14 deg.
75	MD/PA	4 3/4	2 1/2	9/16	1 1/2	2 7/16	13/16	12	VERT.	0.25	0.25	13 deg.	13 deg.
75	Seaboard (Dudley)	5	2 9/16	1/2	1 3/8	2 3/4	7/8	14	14	0.5	0.3125	14 deg.	14 deg.
75	Union Pacific	5	2 9/16	33/64	1 3/8	2 13/16	13/16	12	12	0.25	0.25	13 deg.	13 deg.
75	Union Pacific	4 15/16	2 7/16	33/64	1 3/8	2 5/8	15/16	14	14	0.375	0.375	1 to 4	1 to 4
76	MD/PA	4 3/4	2 1/2	1/2	1 11/16	2 1/4	13/16	20	VERT.	0.3125	0.3125	14° 30'	12 deg.
78	Great Northern	5	2 3/8	5/8	1 11/16	2 1/2	13/16					14 deg.	14 deg.
78	Old Colony	4 3/4	2 1/2	17/32	1 3/4	2 7/32	25/32	12	12	0.4375	0.4375	14 deg.	12° 15'
79	MD/PA	4 3/4	2 5/8	5/8	1 5/8	2 11/32	25/32	12	9	0.25	0.25	13 deg.	13 deg.
80	Frictionless	5 3/16	1 15/16	9/16	2 1/32	2 9/32	7/8					13 deg.	13 deg.
80	ARA-A	5 1/8	2 1/2	33/64	1 7/16	2 23/32	31/32	14	14	0.375	0.375	1 to 4	1 to 4
80	ARA-B	4 15/16	2 7/16	35/64	1 15/32	2 15/32	1	12	12	0.3125	0.3125	13 deg.	13 deg.
80	ASCE	5	2 1/2	35/64	1 1/2	2 5/8	7/8	12	12	0.25	0.25	13 deg.	13 deg.
80	Canadian Northern	5	2 9/16	35/64	1 13/32	2 11/16	29/32					13 deg.	13 deg.
80	Dudley	5 1/8	2 21/32	17/32	1 1/2	2 3/4	7/8	14	14	0.5	0.3125	1 to 4	1 to 4
80	Frictionless	5 3/16	1 15/16	9/16	2 1/32	2 9/32	7/8					13 deg.	13 deg.
80	Great Northern	5	2 13/32	5/8	1 5/8	2 1/2	7/8					14 deg.	14 deg.
80	Hocking Valley	5	2 31/64	29/64	1 3/4	2 25/64	7/8					13 deg.	13 deg.

		Key Rail Dimensions											
		Rail Height	Head Width	Web Thickness	Head Height	Web Height Fishing	Base Height	Head Radius	Web Radius	Top Fillet Radius	Bottom Fillet Radius	Head Bottom Angle	Base Angle
Weight	Type	H	HD	W	HH	WH	BH	RH	RW	R1	R2	A1	A2
80	New York Central	5 1/8	2 21/32	17/32	1 1/2	2 3/4	7/8	14	14	0.5	0.3125	1 to 4	1 to 4
85	ASCE	5 3/16	2 9/16	9/16	1 35/64	2 3/4	57/64	12	12	0.25	0.25	13 deg.	13 deg.
85	C.B. & Q.	5 3/16	2 21/32	9/16	1 35/64	2 3/4	57/64					13 deg.	13 deg.
85	Canadian Pacific	5 1/8	2 1/2	9/16	1 7/16	2 11/16	1	8	8	0.375	0.375	1 to 4	1 to 4
85	Head Free – CP	5 1/4	2 29/64	9/16	1 9/16	2 11/16	1	8	8	0.375	0.375	1 to 4	1 to 4
85	Denver & RG	5 1/4	2 1/2	9/16	1 3/4	2 5/8	7/8					13 deg.	13 deg.
85	D. & R.G. / C & S	5 3/8	2 1/2	9/16	1 15/32	2 29/32	1					1 to 4	1 to 4
85	Great Northern	5	2 21/32	21/32	1 19/32	2 1/2	29/32					14 deg.	14 deg.
85	Missouri Pacific	5 7/32	2 15/32	19/32	1 3/4	2 39/64	55/64					13 deg.	13 deg.
85	N.Y.C. & Stl. / Kcs	5 3/8	2 17/32	17/32	1 29/64	2 15/16	63/64	14	14	0.375	0.375	1 to 4	1 to 4
85	Pennsylvania	5 1/8	2 1/2	17/32	1 21/32	2 15/32	1	10	10	0.25	0.25	15 deg.	13 deg.
85	Pennsylvania	5	2 9/16	17/32	1 3/4	2 3/8	7/8	10	8	0.25	0.25	13 deg.	13 deg.
85	Seaboard (Dudley)	5 1/4	2 11/16	17/32	1 5/8	2 3/4	7/8	14	14	0.5	0.3125	1 to 4	1 to 4
85	Soo Line	5 3/8	2 1/2	9/16	1 15/32	2 29/32	1					14° 2' 11"	14° 2' 11"
85	Western Pacific	5 1/4	2 1/2	9/16	1 3/4	2 5/8	7/8	10	VERT.	0.3125	0.3125	13 deg.	13 deg.
90	ARA-A	5 5/8	2 9/16	9/16	1 15/32	3 5/32	1	14	14	0.375	0.375	1 to 4	1 to 4
90	ARA-B	5 17/64	2 9/16	9/16	1 39/64	2 5/8	1 1/32	12	12	0.3125	0.3125	13 deg.	13 deg.
90	ASCE	5 3/8	2 5/8	9/16	1 19/32	2 55/64	59/64	12	12	0.25	0.25	13 deg.	13 deg.
90	AT & SF	5 5/8	2 9/16	9/16	1 15/32	3 5/32	1					1 to 4	1 to 4
90	Chicago NW	5 17/32	2 1/2	1/2	1 17/32	2 31/32	1 1/32	12	12	0.3125	0.3125	13 deg.	13 deg.
90	Denver & RG	5 1/2	2 9/16	9/16	1 5/8	2 7/8	1					14 deg.	14 deg.
90	Frictionless	5 5/8	2 1/4	9/16	2	2 5/8	1					13 deg.	13 deg.
90	Frictionless	6 3/32	1 59/64	9/16	1 15/16	3 5/32	1					1 to 4	1 to 4
90	Great Northern	5 3/8	2 5/8	9/16	1 15/32	2 7/8	1 1/32	12	14	0.4375	0.625	13 deg.	13 deg.
90	Great Northern	5 3/8	2 5/8	19/32	1 15/32	2 7/8	1 1/32					13 deg.	13 deg.
90	Great Northern	5 3/8	2 5/8	5/8	1 1/2	2 7/8	1	14	14	0.375	0.375	13 deg.	13 deg.
90	Head Free - R.A.	5 25/32	2 31/64	9/16	1 5/8	3 5/32	1	14	14	0.375		1 to 4; U = 54°	1 to 4
90	Interborough R.T.	5	2 7/8	11/16	1 25/32	2 11/32	7/8	12	9	0.25	0.25	13 deg.	13 deg.
90	Lehigh Valley	5	2 3/4	5/8	1 53/64	2 15/64	15/16	12	9	0.25	0.25	14 deg.	14 deg.
90	N.Y.C. (Dudley)	5 1/2	2 21/32	9/16	1 1/2	3 1/32	31/32	14	14	0.5	1	1 to 4	1 to 4
90	Union Pacific	5 3/4	2 3/4	17/32	1 1/2	3 3/8	7/8					13 deg.	13 deg.
91	Lackawanna	5 1/4	2 5/8	5/8	1 41/64	2 11/16	59/64	10	8	0.25	0.25	13 deg.	13 deg.
92	Frictionless	5 7/16	1 15/16	5/8	2 3/32	2 5/16	1 1/32					13 deg.	13 deg.
93	Frictionless	6 1/8	2 1/8	19/32	1 13/16	3 3/8	15/16					13 deg.	13 deg.
95	ASCE	5 9/16	2 11/16	9/16	1 41/64	2 63/64	15/16	12	12	0.25	0.25	13 deg.	13 deg.

		Key Rail Dimensions											
		Rail Height	Head Width	Web Thickness	Head Height	Web Height Fishing	Base Height	Head Radius	Web Radius	Top Fillet Radius	Bottom Fillet Radius	Head Bottom Angle	Base Angle
Weight	Type	H	HD	W	HH	WH	BH	RH	RW	R1	R2	A1	A2
95	Boston & Albany	5 1/32	3	5/8	1 9/16	2 15/32	1	14	14	0.5	0.3125	14 deg.	14 deg.
95	W & H Ry. (Dudley)	5 1/32	3	5/8	1 9/16	2 15/32	1	14	14	0.5	0.3125	1 to 4	1 to 4
97	Frictionless	5 7/8	2 1/4	9/16	1 15/16	2 55/64	1 5/64					13 deg.	13 deg.
98	Frictionless	5 27/32	2 1/2	9/16	1 31/32	2 25/32	1 3/32					15 deg.	13 deg.
100	ARA-A	6	2 3/4	9/16	1 9/16	3 3/8	1 1/16	14	14	0.375	0.375	1 to 4	1 to 4
100	ARA-B	5 41/64	2 21/32	9/16	1 45/64	2 55/64	1 5/64	12	12	0.3125	0.3125	13 deg.	13 deg.
100	AREA	6	2 11/16	9/16	1 21/32	3 9/32	1 1/16	14	14	0.375	0.625	1 to 4	1 to 4
100	ASCE	5 3/4	2 3/4	9/16	1 45/64	3 5/64	31/32	12	12	0.25	0.25	13 deg.	13 deg.
100	Canadian Pacific	6 1/16	2 11/16	9/16	1 23/32	3 9/32	1 1/16	14	14	0.375	0.625	1 to 4	1 to 4
100	Chicago NW	5 45/64	2 9/16	9/16	1 39/64	2 61/64	1 9/64	12	12	0.3125	0.3125	13 deg.	13 deg.
100	Elgin Joliet & East.	5 9/16	2 21/32	9/16	1 37/64	2 51/64	1 3/16					1 to 4	1 to 4
100	Great Northern	5 3/4	2 3/4	9/16	1 5/8	3	1 1/8					1 to 4	1 to 4
100	Head Free - R.A.	6 5/32	2 11/16	9/16	1 23/32	3 3/8	1 1/16	14	14	0.375		1 to 4; U = 49°	1 to 4
100	Head Free - R.E.	6 1/16	2 39/64	9/16	1 23/32	3 9/32	1 1/16					1 to 4; U = 57°	1 to 4
100	Interborough R. T.	5 3/4	2 7/8	9/16	1 45/64	3 5/64	31/32	12	12	0.25	0.25	13 deg.	13 deg.
100	N.Y., N.H. & H.	6	2 3/4	19/32	1 23/32	3 11/32	15/16	12	12	0.25	0.25	13 deg.	13 deg.
100	New York Central	6	3	19/32	1 5/8	3 13/32	31/32	14	14	0.5	0.3125	1 to 4	1 to 4
100	Pennsylvania	5 11/16	2 43/64	9/16	1 13/16	2 25/32	1 3/32	10	10	0.3125	0.3125	15 deg.	13 deg.
100	Pennsylvania	5 1/2	2 13/16	5/8	1 7/8	2 11/16	15/16	10	8	0.25	0.25	13 deg.	13 deg.
100	Reading	5 5/8	2 21/32	9/16	1 45/64	2 55/64	1 1/16	12	12	0.3125	0.3125	13 deg.	13 deg.
100	R.W. Hunt.	6	2 9/16	9/16	1 19/32	3 21/64	1 5/64	12	12	0.375	0.375	14 deg.	14 deg.
101	Lackawanna	5 7/16	2 3/4	5/8	1 23/32	2 11/16	1 1/32	10	8	0.25	0.25	13 deg.	13 deg.
105	Lackawanna	6	2 3/4	5/8	1 23/32	3 1/4	1 1/32	10	8	0.25	0.25	13 deg.	13 deg.
105	Dudley	6	3	5/8	1 5/8	3 13/32	31/32	14	14	0.5	0.75	1 to 4	1 to 4
105	New York Central	6	3	5/8	1 5/8	3 13/32	31/32	14	14	0.5	1	1 to 4	1 to 4
106	Misc.	6 3/16	2 21/32	19/32	1 3/4	3 3/8	1 1/16					1 to 4	1 to 4
107	N.Y., N.H. & H.	6 1/8	2 3/4	19/32	1 23/32	3 11/32	1 1/16	12	12	0.25	0.25	13 deg.	13 deg.
110	AREA	6 1/4	2 25/32	19/32	1 23/32	3 13/32	1 1/8	14	14	0.375	0.625	1 to 4	1 to 4
110	ASCE	6 1/8	2 7/8	37/64	1 25/32	3 11/32	1	12	12	0.25	0.25	13 deg.	13 deg.
110	C.T.A.	7	2 3/4	9/16	1 7/8	4 5/16	13/16					14 deg.	9 deg.
110	Great Northern	6 1/2	2 3/4	19/32	1 5/8	3 3/4	1 1/8	14	14	0.5	0.625	1 to 4	1 to 4
110	Head Free - AREA	6 7/16	2 11/16	19/32	1 29/32	3 13/32	1 1/8	14	14	0.375		1 to 4; U = 55° 30'	1 to 4
110	Lehigh Valley	6	2 7/8	19/32	1 7/8	3 1/16	1 1/16					1 to 4	1 to 4
112	AREA	6 5/8	2 23/32	19/32	1 11/16	3 13/16	1 1/8	24	10 & 23	0.375	0.625	1 to 4	1 to 4
112	Head Free - R.E.	6 3/4	2 11/16	19/32	1 13/16	3 13/16	1 1/8	14	10 & 23	0.375		1 to 4; U = 58°	1 to 4

		Key Rail Dimensions											
		Rail Height	Head Width	Web Thickness	Head Height	Web Height Fishing	Base Height	Head Radius	Web Radius	Top Fillet Radius	Bottom Fillet Radius	Head Bottom Angle	Base Angle
Weight	Type	H	HD	W	HH	WH	BH	RH	RW	R1	R2	A1	A2
112	CB & Q – TR	6 3/4	2 1/2	5/8	1 3/4	3 7/8	1 1/8					1 to 4; U = 77° 45'	1 to 4
113	Head Free – SP	6 13/16	2 11/16	19/32	1 7/8	3 13/16	1 1/8	14	10 & 23	0.375		1 to 4; U = 58°	1 to 4
115	AREA	6 5/8	2 23/32	5/8	1 11/16	3 13/16	1 1/8	10	3 & 14	0.75	0.75	1 to 4	1 to 4
115	D.R.G.W.	6 5/8	2 23/32	3/4	1 11/16	3 13/16	1 1/8					13 deg.	13 deg.
115	Dudley	6 1/2	3	5/8	1 11/16	3 3/4	1 1/16	14	14	0.5	0.75	1 to 4	1 to 4
115	Miscellaneous	6	2 15/16	21/32	1 7/8	3 1/16	1 1/16					1 to 4	1 to 4
118	Lackawanna	6 1/2	2 7/8	5/8	1 29/32	3 1/2	1 3/32					13 deg.	13 deg.
119	AREA	6 13/16	2 21/32	5/8	1 7/8	3 13/16	1 1/8	14	3 & 14	0.75	0.75	1 to 4	1 to 4
120	AREA	6 1/2	2 7/8	5/8	1 25/32	3 17/32	1 3/16					1 to 4	1 to 4
120	Mfg. Std.	6 1/4	2 7/8	5/8	1 29/32	3 5/32	1 3/16	12	12	0.375	0.375	14 deg.	14 deg.
120	New York Central	7	3	21/32	1 5/8	4 5/16	1 1/16	14	20	0.5	1	1 to 4	1 to 4
122	CB (B&O)	6 25/32	2 15/16	21/32	1 15/16	3 39/64	1 15/64	10	3 & 14	0.75	0.75	1 to 2 3/4	1 to 2 3/4, 1 to 13.7
125	Pennsylvania	6 1/2	3	21/32	1 7/8	3 13/32	1 7/32	12	16	0.5	0.75	18 deg.	14 deg.
126	Frictionless	7	1 13/16	11/16	2 3/8	3 13/32	1 7/32					18 deg.	14 deg.
127	Dudley	7	3	21/32	1 11/16	4 5/32	1 5/32					1 to 4	1 to 4
127	New York Central	7	3	21/32	1 11/16	4 5/32	1 5/32	14	18	0.5	0.75	1 to 4	1 to 4
129	CB & Q – TR	7 5/16	2 5/8	21/32	1 27/32	4 9/32	1 3/16					1 to 4	1 to 4
130	AREA	6 3/4	2 15/16	21/32	1 27/32	3 11/16	1 7/32	14	14	0.5	0.75	1 to 4	1 to 4
130	Head Free - P.S.	6 13/16	3	21/32	2 3/16	3 3/8	1 7/32					18°; U = 58° 30'	14 deg.
130	Head Free - R.E.	6 13/16	2 27/32	21/32	2 1/32	3 11/16	1 7/32	14	14	0.5		1 to 4; U = 61°	1 to 4
130	Phil. & Reading	6 27/32	2 15/16	21/32	1 15/16	3 11/16	1 7/32					1 to 4	1 to 4
130	Pennsylvania	6 5/8	3	11/16	2	3 13/32	1 7/32	12	16	0.5	0.75	18 deg.	14 deg.
131	AREA	7 1/8	3	21/32	1 3/4	4 3/16	1 3/16	24	10 & 23	0.5	0.75	1 to 4	1 to 4
131	Head Free - R.E.	7 1/4	2 31/32	21/32	1 7/8	4 3/16	1 3/16	14	10 & 23	0.5		1 to 4; U = 60° 30'	1 to 4
132	AREA	7 1/8	3	21/32	1 3/4	4 3/16	1 3/16	10	8 & 16	3/4 & 5/16	0.875	1 to 4	1 to 4
132	Head Free – S.P.	7 5/16	2 31/32	21/32	1 15/16	4 3/16	1 3/16	14	10 & 23	0.5		1 to 4; U = 60° 30'	1 to 4
133	AREA	7 1/16	3	11/16	1 15/16	3 15/16	1 3/16	10	8 & 16	3/4 & 7/16	0.75	1 to 3	1 to 4.011
135	Central of NJ	6 1/2	3 5/32	3/4	2	3 9/32	1 7/32					14 deg.	14 deg.
136	AREA	7 5/16	2 15/16	11/16	1 15/16	4 3/16	1 3/16	14	8 & 20	3/4 & 5/16	0.75	1 to 4	1 to 4
136	Lehigh Valley	7	2 15/16	21/32	1 7/8	3 7/8	1 1/4					1 to 4	1 to 4
136	Lehigh Valley	7 3/8	2 15/16	11/16	1 25/32	4 3/8	1 7/32					1 to 4	1 to 4
136	Lehigh Valley	7	2 15/16	11/16	1 7/8	3 7/8	1 1/4					1 to 4	1 to 4
136	New York Central	7 9/32	2 15/16	11/16	1 7/8	4 5/32							
140	AREA/PS	7 5/16	3	3/4	2 1/16	4 1/16	1 3/16	10	8 & 27	3/4 & 7/16	0.75	1 to 3	1 to 4
141	AREA	7 7/16	3 1/16	11/16	2 5/32	4 3/32	1 3/16		19 31/32	¾	3/4	18.4 deg.	14 deg.

		Key Rail Dimensions											
		Rail Height	Head Width	Web Thickness	Head Height	Web Height Fishing	Base Height	Head Radius	Web Radius	Top Fillet Radius	Bottom Fillet Radius	Head Bottom Angle	Base Angle
Weight	Type	H	HD	W	HH	WH	BH	RH	RW	R1	R2	A1	A2
152	Pennsylvania	8	3	11/16	1 27/32	4 7/8	1 9/32	24	6&30	0.5	0.75	14 deg.	14 deg.
155	Pennsylvania	8	3	3/4	2 1/16	4 21/32	1 9/32					18° 26' 10"	14 deg.



Appendix D – Use of portable track-loading fixture (PTLF) in non-GRMS territory

Note – The use of the PTLF for compliance purposes outside GRMS territory has been temporality suspended.

Appendix E – CWR Joint Bar Fracture Report

(For Reference Purposes Only – Please use the official form available at: http://safetydata.fra.dot.gov/CWR/.)

CWR JOINT BAR FRACTURE REPORT	***TYPE OF INSPECTION*** ☐ PERIODIC JOINT INSPECTION (213.119[h][6][i]) ☐ TRACK INSPECTION (213.233) ☐ TURNOUT INSPECTION (213.235) ☐ OTHER (discovered during other than required inspection)		
RAILROAD:	SUBDIVISION:		MILEPOST:
DATE FOUND: ____ /____ 20____	ANNUAL MGT:	TRACK #:	TRACK CLASS:
☐ TANGENT	☐ CURVE ____ degrees ☐ IN SPIRAL	☐ LOW/INNER RAIL ☐ HIGH/OUTER RAIL	RAIL SECTION(S):______ / ______
ANNUAL JOINT INSPECTION FREQUENCY FOR THIS SEGMENT ☐ 1x ☐ 2x ☐ 3x ☐ 4x ☐ OTHER: ____________		DATE OF LAST JOINT INSPECTION: ____ /____ /20____	

BAR TYPE (check all that apply)	☐ STANDARD ☐ INSULATED ☐ COMPROMISE NUMBER OF HOLES: ☐ 4 ☐ 5 ☐ 6 ☐ 7 ☐ 8

FIELD SIDE BAR	***GAGE SIDE BAR***
BROKEN THROUGH Check location of break: ☐CENTER ☐ INNER BOLT HOLE ☐ OTHER	**BROKEN THROUGH** Check location of break: ☐CENTER ☐ INNER BOLT HOLE ☐ OTHER
CRACKED Check location(s) and record length(s): ☐ TOP CENTER ________ inches ☐ BOTTOM CENTER ________ inches ☐ INNER BOLT HOLE ________ inches ☐ OTHER BOLT HOLE ________ inches ☐ OTHER (describe) ________ inches	**CRACKED** Check location(s) and record length(s): ☐ TOP CENTER ________ inches ☐ BOTTOM CENTER ________ inches ☐ INNER BOLT HOLE ________ inches ☐ OTHER BOLT HOLE ________ inches ☐ OTHER (describe) ________ inches

GAP BETWEEN RAIL ENDS _______ INCHES
RAIL END BATTER OR RAMP (Figures 1 and 2)
☐ NORTH or ☐ EAST RAIL END _______ INCHES HIGH _______ INCHES LONG
☐ SOUTH or ☐ WEST RAIL END_______ INCHES HIGH _______ INCHES LONG
TREAD MISMATCH _______ INCHES (Figure 3)
JOINT VERTICAL MOVEMENT _______ INCHES

<u>IF JOINT IN CURVE or SPIRAL:</u>
GAGE RAMP (Figure 4) _______ INCHES OUT _______ INCHES LONG
GAGE MISMATCH (Figure 5) _______ INCHES
JOINT LATERAL MOVEMENT _______ INCHES

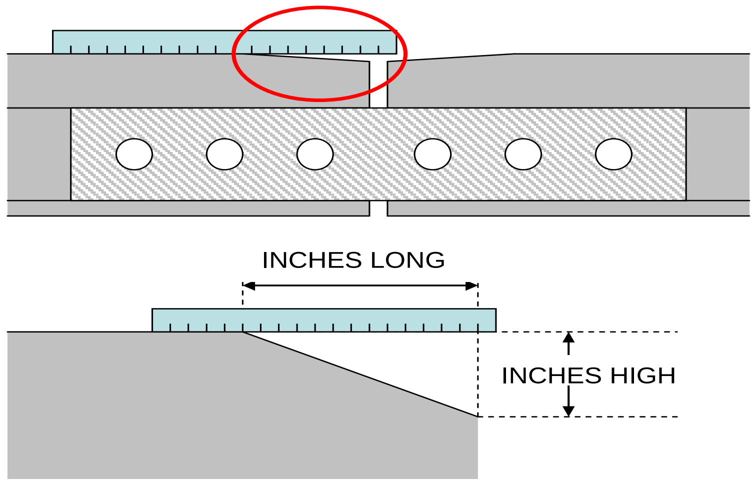
OTHER COMMENTS:

FRACTURE REPORT INSTRUCTIONS

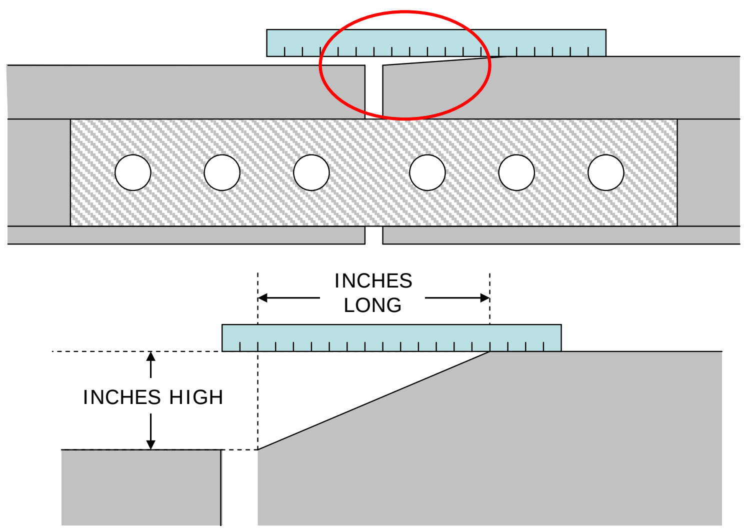
TYPE OF INSPECTION – Indicate the type of inspection being performed when fracture was found. At least one (1) box in group must be checked.
RAILROAD – FRA railroad reporting code, (*e.g.* CSX or NS). Four (4) character alpha.
SUBDIVISION – Railroad's subdivision or district. If none enter “system”. Fourteen (14) character alphanumeric.[2]
MILEPOST – Railroad's designated milepost at the location of the fracture. 7.2 character alphanumeric, *e.g.*, ABC1234.56.[1]
DATE FOUND – Date the fracture was found. Eight (8) character numeric, MMDDYYYY.
ANNUAL MGT – Million Gross Tons (from previous year) for the specific track with the fracture. 4.1 numeric, *e.g.*, 123.4 (allowable range 0 to 999.9 inclusive).
TRACK CLASS – FRA Class for track with the fracture. One (1) character numeric, *e.g.*, 3 (allowable range 2 - 6 inclusive).
TANGENT/CURVE/SPIRAL/INNER/OUTER – Indicate whether fracture found on tangent, curve (include degree of curvature) or spiral and if inner or outer rail, if applicable. If tangent, check TANGENT. Otherwise check CURVE or SPIRAL and INNER or OUTER. If curve checked, curvature entered as 2.1 numeric, *e.g.* 2.5.
RAIL SECTION – Indicate each rail section comprising the joint, (*e.g.* for a standard bar, enter 136 or for a compromise bar, enter 132/115).
ANNUAL JOINT INSPECTION FREQUENCY – Number of times per year that walking joint bar inspection is performed. Two (2) character numeric, *e.g.* 3 (allowable range 1 – 12 inclusive).
DATE OF LAST JOINT BAR INSPECTION – Date the last walking joint bar inspection was performed. Eight (8) character numeric, MMDDYYYY.
BAR TYPE/HOLES – Indicate bar type: standard, insulated, or compromise bar and number of holes. Two (2) boxes (one in each group) must be checked.
BROKEN THROUGH – For each bar, field and gage, check appropriate box if broken completely through and indicate the location of the break (through center, through inner bolt hole or other location). For each bar, field and gage, there is no requirement to check any box(es) – neither bar is broken through.
CRACKED – For each bar, field and gage, indicate the crack location(s) and corresponding length(s). For each bar, field and gage, any number of boxes may be checked. If box is checked, crack length is 3.1 numeric, *e.g.*, 2.5. If OTHER is checked, text description can be 64 (128) character alpha-numeric.
GAP BETWEEN RAIL ENDS – Measure and record the distance between the rail ends. If joint is pulled apart or separated, estimate the gap prior to separation. 5.2 numeric, *e.g.* 10.25.
RAIL END BATTER OR RAMP - Measure and record the ***height and length of the batter or ramp for each rail end*** and record even if found to be zero. See Figures 1 and 2 for method of measurement. Check appropriate boxes (one each of NORTH or EAST and one each of SOUTH or WEST) and enter batter ramp as four (4) 4.2 numeric, *e.g.*, 1.25.

[2] This format has been pre-established in FRA's RISPC system for its safety inspectors.

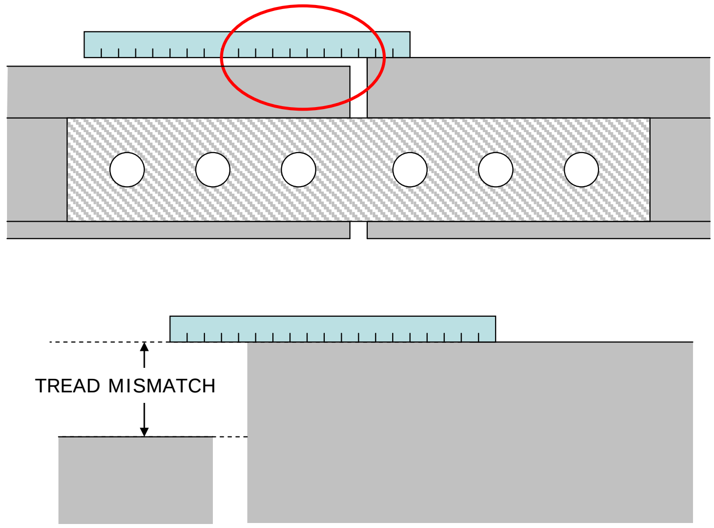
TREAD MISMATCH – Measure and record the tread mismatch. See Figure 3 for method of measurement. 4.2 numeric, *e.g.*, 1.25.
JOINT VERTICAL MOVEMENT – Record the vertical movement of the rail joint (not track surface) according to 213.13. 4.2 numeric, *e.g.*, 1.25.
GAGE RAMP – In curves only, measure and record the gage ramp distance out and length. See Figure 4 for method of measurement. Two (2) 4.2 numeric, *e.g.*, 1.25.
GAGE MISMATCH – In curves only, measure and record the gage mismatch. See Figure 5 for method of measurement. 4.2 numeric, *e.g.*, 1.25.
JOINT LATERAL MOVEMENT – In curves only, record the lateral movement of the rail joint (not gage) according to 213.13. 4.2 numeric, *e.g.*, 1.25.
OTHER COMMENTS: - Other comments, including any other factors or conditions that may have contributed to the fracture of the bar(s). 256 character alphanumeric.



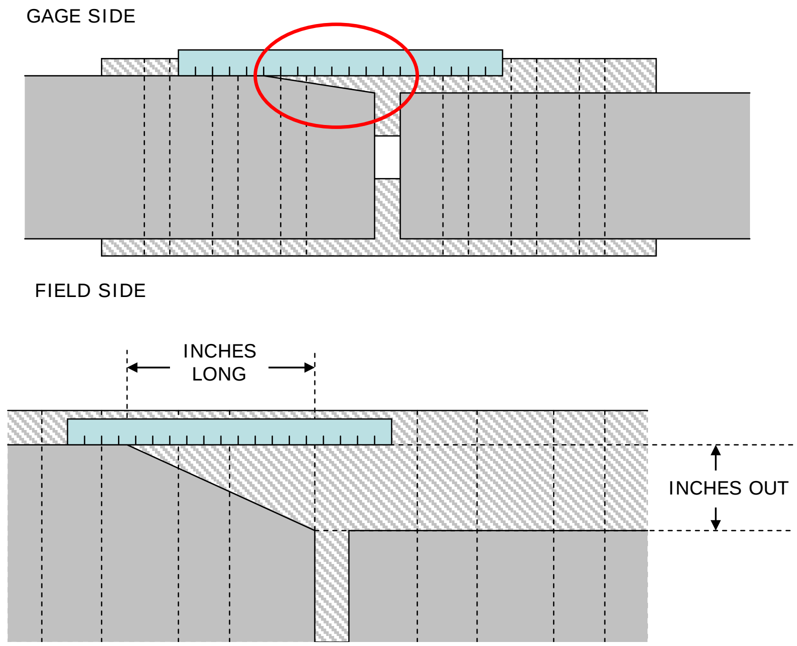
CWR Joint Bar Fracture Report - Figure 1
Method for measuring RAIL END BATTER.
Measurement to be made on each rail end.
(NOT TO SCALE)



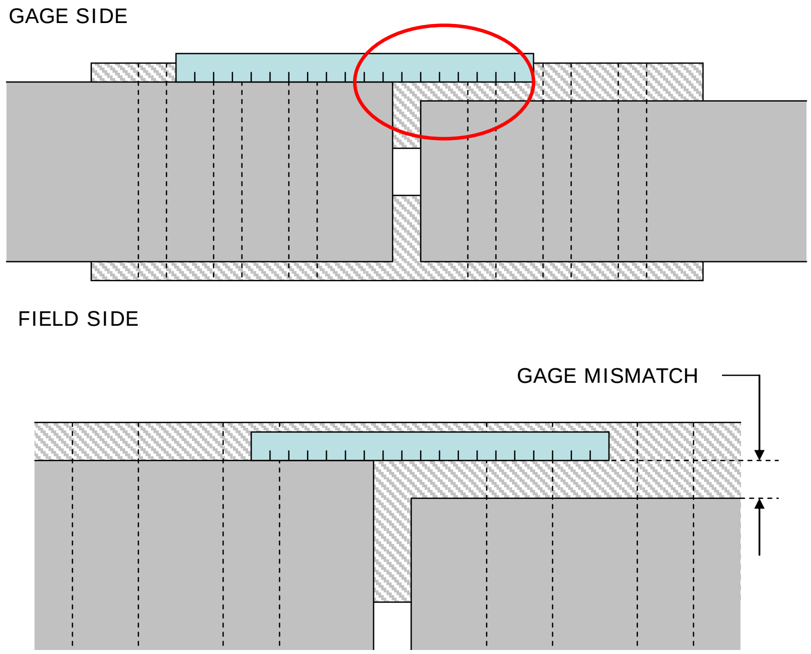
CWR Joint Bar Fracture Report - Figure 2.
Method for measuring RAIL END RAMP.
(NOT TO SCALE)



CWR Joint Bar Fracture Report - Figure 3
Method for measuring TREAD MISMATCH.
(NOT TO SCALE)



CWR Joint Bar Fracture Report - Figure 4
Method for measuring GAGE RAMP.
(NOT TO SCALE)



CWR Joint Bar Fracture Report - Figure 5.
Method for measuring GAGE MISMATCH.
(NOT TO SCALE)

NOTICE OF SAFETY ADVISORY 97-01 - Damage to tracks, roadbed, and bridges resulting from uncontrolled flows of water and similar weather-related phenomena.

On September 4, 1997, FRA published Notice of Safety Advisory 97-1 in the Federal Register (Vol. 62, No. 171), addressing safety practices to reduce the risk of casualties from train derailments caused by damage to tracks, roadbed, and bridges resulting from uncontrolled flows of water and similar weather-related phenomena. This was amended on November 14, 1997 (Vol. 62, No. 220) by revising the recommendation concerning the transmission of flash flood warnings to train dispatchers or other employees controlling the movement of trains.

A recent derailment involving train number 4 of the National Railroad Passenger Corporation (Amtrak) on the Burlington Northern and Santa Fe Railway Company (BNSF) near Kingman, Arizona, on August 9, 1997, has caused FRA to focus on the effectiveness of railroad procedures for protection of trains and personnel from hazards caused by severe weather conditions. The investigation of that accident by the National Transportation Safety Board (NTSB) and FRA continues. The facts and findings developed in the investigation will be published when the investigation is complete.

Special Inspection Procedures: The Federal Track Safety Standards (49 CFR part 213) state, "In the event of fire, flood, severe storm, or other occurrence which might have damaged track structure, a special inspection must be made of the track involved as soon as possible after the occurrence ." (49 CFR 213.239). This provision is purposely general in nature, because it is not practicable to specify in a minimum safety standard all the conditions which could trigger a special inspection, nor the manner in which any particular special inspection must be conducted. However, in accordance with the primary purpose of the Track Safety Standards and associated railroad safety laws, these special inspections should be conducted so as to effectively prevent derailments. In light of recent occurrences and past experience, FRA now believes it necessary to issue this safety advisory to provide railroads with recommended procedures to further this objective.

FRA has investigated several train derailments in which damage from unexpected moving water was a causal factor. Incidents reported to FRA between January 1982 and March 1996 included 26 derailments caused by washouts of bridges or bridge approaches, and 16 derailments caused by washouts or water damage to culverts or subgrade not near a bridge. In most cases, the railroad had some form of notification of the severe weather conditions and either initiated or performed an inspection. When the derailment occurred, either the inspector had not reached the derailment site before the train, had inspected the track and not recognized a hazardous condition, or had performed the inspection before the damage had become detectible. FRA believes that more specific measures can be taken by each railroad that conducts operations on track subject to hazards from flowing water, to reduce the likelihood of future derailments caused by those hazards.

Vulnerable Structures and Track: FRA believes that several types of bridge and drainage structure components should be identified as vulnerable and be given special consideration in

any decision related to the operation of trains both during and following a severe rainstorm. In particular, bents, piers, and abutments that rest directly on soil or degradable rock near the surface might be rapidly undermined in a severe rainstorm. Similarly, stream bed configurations in which the water course takes a bend or a change in slope near the track are often unpredictable in times of heavy flow. During such conditions, soil displacement can progress rapidly in an unpredictable manner in locations that are not visible to a person above the water surface. The size of a drainage structure, and whether it is categorized as a bridge or a culvert, is not as important as the vulnerability of the structure and its supported track to the effects of flowing water.

Recommended Action: FRA believes that the chance of further derailments, such as occurred near Kingman, Arizona, on August 9, 1997, would be greatly reduced by the inclusion of certain additional measures into the procedures for special inspections followed in the railroad industry in the event of a threat of a severe rainstorm, at the level of a flash flood. FRA has determined that each railroad that controls the operation of trains on Class 4 or higher track, or passenger trains in commuter or intercity service, should have in place a program to protect its train operations from the effects of damage to tracks and structures caused by severe weather conditions, particularly flash floods. Therefore, FRA issues the following advisory to each affected railroad:

1. The railroad should have in place a procedure that will assure that the train dispatchers or other employees controlling the movement of trains on all track of Class 4 or higher or upon which passenger trains operate in commuter or intercity service will receive timely warnings of any flash flood that might damage that track or its supporting structures. In the case of such track located outside of the warning area but subject to damage from water resulting from the storm, the information should be obtained in time to permit timely response by the railroad. The warning procedure should incorporate either:

a. The means to receive within 15 minutes of issuance by the National Weather Service (NWS) all NWS flood warnings for the area in which the track is located; or

b. An arrangement with a competent commercial weather service which receives and reviews warnings and weather data from the NWS as part of its warning procedures, and from which the railroad receives warnings and weather information that is specific to the situation and requirements of the railroad.

2. After the receipt of a warning of a flash flood which might damage track or bridges, the railroad should notify train crews and limit the speed of all freight and passenger trains to that which will permit the train to operate safely, consistent with the potential water levels and visibility conditions, on all track subject to damage from the flood. The limitations should continue until a special inspection in accordance with 49 CFR 213.239 has been performed of that track and it is determined that a hazard no longer exists. In making that inspection and determination, the time taken for the heaviest flow of water to reach the track should be considered.

3. Each railroad affected by this advisory should identify its bridges carrying track of Class

4 or higher or over which passenger trains operate in commuter or intercity service, which are vulnerable to damage from flash floods or similar weather-related phenomena. Particular attention should be given to bridges which incorporate piers, bents, or abutments, which are founded on soil or degradable rock which could lose its integrity as a result of scour by moving water, and which are commonly referred to as "mud sills" or "spread footings."

4. The information developed in paragraph 3 should be compiled and made available to each person who can be called upon to perform special inspections on the subject track following a flash flood warning. Consideration should be given to placing identifying marks on bridges that need particular attention in special inspections, along with the bridge number, to assist inspectors in locating them with certainty during inclement weather. Consideration should also be given to the use of automated high water detectors or similar sensing and warning systems on specific bridges which could incur water damage that would be hidden from or not otherwise detectible by a human inspector.

5. In addition to the bridge-specific information called for in paragraph 3, each affected railroad should implement a training program for the persons performing special inspections. The training should include methods to recognize and protect the safety of railroad operations from the damaging characteristics of flowing water in general, with particular regard to the effects of a watercourse that takes a significant change in horizontal direction or vertical profile near the track; the effects of drift material accumulation on scour and the capacity of the waterway opening; and the potential for damage by impact of heavy floating objects.

6. Refresher training of track inspectors on the subjects addressed in paragraph 5 should be conducted at least once each calendar year. Where practicable, that refresher training should include a joint inspection by a track inspector and a cognizant bridge maintenance or engineering employee over the inspector's assigned territory. During that joint inspection they should locate the vulnerable components in the bridges identified in paragraph 3, discuss the precautions to be taken in the event of indications of distress in those components, observe drainage conditions on and adjacent to the right-of-way, and note changes for inclusion in the revisions of information called for in paragraph 9.

7. If a track inspector is assigned to perform a special inspection in accordance with paragraph 2, and bridges identified as vulnerable are in the track segment subject to damage from the flash flood, a cognizant bridge maintenance or engineering employee should be readily available by telephone or radio to assist in the interpretation of findings by the track inspector.

8. Each affected railroad should brief all of its track and bridge inspectors on the contents of this advisory. These briefings should occur within 14 calendar days of the date of publication of this safety advisory in the Federal Register.

9. FRA believes that the actions described in paragraphs 3, 4, and 5 should be completed within 60 calendar days of the date of publication of this safety advisory in the Federal Register. During this period, each affected railroad should complete an initial review of its bridges for vulnerability to high or rapidly flowing water and provide that information to its inspectors.

More detailed reviews should be substantially completed and provided to inspectors during calendar year 1998 and then maintained in a current status.

10. FRA requests a letter within 45 calendar days of the date of publication of this safety advisory in the Federal Register from each affected railroad specifying the actions it has taken and will initiate to enhance the safety of train operations in the event of a flood or a high or rapid water condition. Such letters should be addressed to the Associate Administrator for Safety, FRA, RRS-1, Mail Stop 25, 400 Seventh Street, S.W., Washington, DC 20590.

Notice of Safety Advisory Amendment: FRA is amending Safety Advisory 97-1, which addresses safety practices to reduce the risk of casualties from train derailments caused by damage to tracks, roadbed, and bridges resulting from uncontrolled flows of water and similar weather-related phenomena, by revising the recommendation concerning the transmission of flash flood warnings to train dispatchers or other employees controlling the movement of trains.

On September 4, 1997, FRA issued Safety Advisory 97-1, recommending that railroads take certain actions to reduce the risk of train derailments which could result from severe weather conditions, particularly undetected flash floods. The first recommendation of SA 97-1 reads as follows:

1. The railroad should have in place a procedure that will assure that all notifications issued by the National Weather Service (NWS) of flash flood warnings will be received within 15 minutes of issuance from the NWS, directly or through a contract weather forecasting service, by the train dispatchers or other employees controlling the movement of trains on all track of Class 4 or higher or upon which passenger trains operate in commuter or intercity service, within the warning area. In the case of such track located outside of the warning area but subject to damage from water resulting from the storm, the information should be obtained in time to permit timely response by the railroad.

The intent of the recommendation is for all flash flood warnings issued by the NWS for the area in which an affected railroad operates to be received by the personnel who control train operations in the area of the warning. It is not necessary that the warning come directly from the NWS, but it should be received intact and in a timely manner.

Since SA 97-1 was issued, FRA has become aware of several circumstances in which large railroads with centralized dispatching operations have contracted with specialized weather services for weather information tailored to the situation and requirements of the railroad. Several of those contract services do not pass on all NWS warnings, but instead analyze the warnings in the light of other weather data available to them and their knowledge of the specific situation and requirements of their clients in order to provide only the weather information that affects the client and to filter out irrelevant information. This process reduces the amount of information that the client is required to consider and evaluate, and allows the client to focus on information that, in the view of the contract weather service, might actually affect the client's property and operations.

FRA now believes that this procedure offered by contract weather services might meet the requirements of some railroads better than if all NWS warnings are passed on by the contract weather service en masse, regardless of their relevance to the individual railroad. Therefore, SA 97-01 is amended in part by revising Recommendation 1.

Paperwork Reduction Act Provisions: This advisory does not require that any records or reports be kept or submitted. It merely recommends that railroads collect or provide certain information. Nevertheless, because some might see these recommendations as paperwork burdens, FRA will seek approval of them....See Federal Register notice for additional language on the paperwork reduction act provisions...

FRA may modify Safety Advisory 97-1, issue additional safety advisories, or take other appropriate necessary action to ensure the highest level of safety on the Nation's railroads.

Issued in Washington, DC, on September 2, 1997. James T. Schultz, Associate Administrator for Safety. The amendment was issued in Washington, DC, on November 10, 1997. George A. Gavalla, Acting Associate Administrator for Safety.

AREMA: Manual for Railway Engineering

3.1.2 IMPORTANCE (1992)

Properly designed openings, control of flood flows, and protection of roadway and structures are of vast importance from the standpoints of safety, economy, and continuity of operation during flood periods. With the
ever-present menace of floods and then disastrous consequences, every related problem is deserving of accurate
and exhaustive survey and careful planning.

SECTION 3.2 DRAINAGE BASIN DATA[1]

3.2.1 GENERAL (1992)

a. Survey requirements depend in some degree upon whether the waterways are to be crossed by a new line, or whether the replacement of an existing waterway structure is involved.

b. For the crossing of a new line the survey requirements are extensive and general in nature, involving the determination of the drainage area and its shape; the stream and slope profile; soil, vegetation and climatic characteristics; as well as topographical details in the vicinity of the most probable point of crossing.

c. For the replacement of an existing waterway structure the survey requirements may be the same as those for a new line, but in many cases the required waterway area will likely be determined from past performance of the stream at the structure to be replaced. Observation may have indicated that a change in size, shape or location of the waterway structure may be desirable. Maintenance of railroad operation during construction and how well the existing structure fits into the local topography may control the design of the new structure. All such facts should be considered in determining survey requirements.

d. Consideration should always be given to probable future changes in conditions above and below the point of crossing which would in any way affect the performance of the stream – for example, channel improvements and the construction or removal of dams or revetments. Also, a search should be made for future subdivision or commercial development plans.

e. For small culverts or replacements some of the data listed here may be unnecessary and some will have been predetermined, but all of the following items should be considered in order that survey notes may include all the information necessary for the design of the most suitable structure:

(1) Area of drainage basin.
(2) Shape and contour of drainage basin.
(3) Location, length and slope of defined channels.
(4) Slope of stream bed and side slopes.
(5) Character of soil and subsoil.
(6) Vegetation – timber, grass, cultivated or barren, and probable changes.

[1] References, Vol. 39, 1938, pp. 322, 786; Vol. 51, 1950, pp. 706, 839; Vol. 54, 1953, pp. 1087, 1385; Vol. 62, 1961, pp. 678, 936; Vol. 73, 1972,

(7) Climatic conditions – accumulation of snow and ice.
(8) Precipitation – local records, if any, of intensity, duration, frequency, and temporal and area distribution.
(9) Natural and artificial storage – lakes, swamps, reservoirs.
(10) Channel course – fixed or changeable.
(11) Channel material – rock, boulders, gravel, sand, clay, silt.
(12) Channel erosion – amount and nature of material transported.
(13) Possibility of ice gorges, or drift accumulations.
(14) Elevation of backwater from larger stream below crossing.
(15) Determination of past flood crests and frequency. On an existing line crossing a wide valley with two or more openings secure high water profile across the valley on both sides of the railroad embankment.
(16) Character of current – rate of flow – steady or variable.

(17) Waterway area, relative flood flows and adequacy of existing drainage structures nearby on the waterway.
(18) Topography over liberal area in vicinity of crossing. Typical flood channel sections.
(19) Location of right-of-way limits.
(20) Property lines and owners names along stream if channel change is contemplated.
(21) Track profile, alignment and topography for sufficient distance to cover any probable change, or as necessary to portray conditions.
(22) Borings locate and give character of material found.
(23) Determine most favorable angle of stream crossing.
(24) Location – mile post and survey station.
(25) Location of borrow pit if bridge filling is involved.
(26) Location and elevation of improvements which might be subject to flooding by backwater upstream from track.
(27) Flood plain regulations and studies, if available.
(28) Governmental regulations and requirements.

SECTION 3.3 CAPACITY OF WATERWAY OPENINGS[1]

3.3.1 GENERAL (1984)

a. In the past, in the design of drainage structures, it was considered sufficient to provide a waterway opening of a certain area, based on an area formula (e.g. Talbot's formula); also, in case the flow (Q) were known, one could assume a velocity of flow (V) (usually taken as 10 feet per sec) and, thus, arrive at the required area (A) of opening. Modern practice is to first calculate the drainage, or flow, then to design the structure to accommodate the flow (Reference 6) using the principles of hydraulics.
b. Before deciding on the hydraulic capacity to be provided in a structure, it is advisable to make a thorough search to determine what precipitation and stream flow records are available in the general region of the project site. Where data and time are available, several methods of determining the required capacity should be used, and their results compared before a decision is made. Extensive study and research are in continuous progress by various public agencies in the field of flood runoff and waterway requirements, and it is expected that much additional useful data will be developed.
c. These agencies are much better suited, both in full-time personnel in this specialized field and in access to pertinent data as quickly as it becomes available, than are most railroads. Therefore, the needs of railroad personnel dealing with drainage matters are best served by having at hand a list of agencies through which they can obtain the latest information on the subject. In order to take full advantage of this and to insure uniformity of design criteria on the individual railroad, it is advisable that all drainage recommendations and supporting data should clear through a designated "drainage engineer" before final decision.
d. A list of Federal and State agencies in the United States that are engaged in research, accumulation of data, and the statistical analysis of precipitation and runoff is given below:

• **Federal**

United States Geological Survey
Water Supply Papers
Flood Magnitude-Frequency Reports
Federal Highway Administration
Transportation Research Board
Soil Conservation Service
Corps of Engineers, U.S. Army
Water and Power Resources Service
National Oceanic and Atmospheric Administration

• **State**

Highway Departments
Water Resources Department
Public Works Departments
Universities
• **County or Parish**
Highway Departments
Public Works Departments
In Canada, stream flow data can be secured from the Water Survey of Canada Division of Environment Canada at Ottawa. Stream flow data are, generally, not available for Mexico.

1 References, Vol. 42, 1941, pp. 543, 831; Vol. 53, 1952, pp. 699, 1106; Vol. 54, 1953, pp. 1087, 1385; Vol. 62, 1961, pp. 678, 936; Vol. 73, 1972, p. 144; Vol. 85, 1984, p. 5.

3.3.2 METHODS (1984)

3.3.2.1 General

The recommended methods will be those most easily applied that give the best results with a minimum of available information. The basic data required are covered in Section 3.2, Drainage Basin Data. In addition, an indispensable aid is a collection of topographic maps as published by the U.S. Geological Survey in conjunction
with TVA, Mississippi River Commission, U.S. Corps of Engineers, etc. With these data in hand, the engineer can proceed to specific methods. These methods are presented in the sections which follow.

3.3.2.2 Statistical Methods

a. When there exist sufficient flow data for the waterway under consideration, a statistical analysis can be undertaken to estimate the probability that a given magnitude of flow can occur or be exceeded (Reference 13). Federal regulations mandate the use of the Log Pearson Type III distribution, a procedure for analyzing extreme events such as maximum yearly discharges for a stream.

b. Special plotting paper (probability × 2 log cycles) can be used for this analysis by following the steps listed below:

(1) Collect runoff data from a stream gaging station near to the desired location, as obtained from such as the U.S. Geological Survey Water Supply Papers. A usable record must have a continuity of data of 20 years or more.
(2) Arrange the data by listing in descending order the magnitudes of the largest recorded peak discharges, i.e. the largest first. The ranking is only by discharge quantity and is independent of when the discharge occurred in the period of record. The series may be terminated when the number of peak discharges equals the number of years of record. The peak discharges should then be numbered (labelled) from one to the number of years of record.
(3) Calculate the exceedance probability by first calculating the estimated return period for each peak discharge by

EQ 1

where:
The exceedance probability
and represents the probability that the specific peak discharge will occur or beexceeded
(4) Utilizing the plotting paper, select a suitable vertical scale so all the discharge data can be graphed and still allow predicted values to be read. Plot the data of peak discharges on the vertical axis and the exceedance probability (%) on the horizontal axis. Fit the best straight line to the plotted data. As an example, using Figure 1-3-1, the probability that a discharge of 10,000 c.f.s. will be equalled or exceeded is 10.9% (i.e. a return period of 9.1 years) for this record. Again, using Figure 1-3-1, the 100-year discharge (100 year return period or 1% exceedance probability) is 15,800 c.f.s.
c. The design flood frequency to be used is a matter of engineering judgment and economics. A number of trials should be made using a wide range of frequencies. In this way the possibilities of damage because of too small an opening can be assessed. The cost of providing for the maximum possible flood of 100 year
n = number of years of record
m = rank or order number of that particular peak discharge

$$T_r = \frac{(n + 1)}{m}$$

p 1
Tr
= ------

frequency or greater can also be determined and a prudent decision made. In general practice, railroad drainage openings should be designed for floods in the range of 25 to 50 years. This does not imply that a 100-year flood design would be out of place in certain instances. Because of the susceptibility of railroads to legal action for damages, it would probably be unwise to design for less than a 25-year flood, except in special instances where results of lesser design are fully understood.

d. After the design flow in cubic feet per second has been determined, the basic hydraulic formula Q = AV can be used to determine the average velocity in feet per second through a given area of opening in square feet. For structures on unstable soils, 3 feet per sec may be the maximum allowable velocity without damaging scour; generally, 3 to 6 feet per sec will cause little, if any, scour in fairly good soils. Culverts and other paved waterways are frequently designed for flows as high as 10 feet per sec. If time permits and greater refinement is desired, there is a multitude of hydraulics texts and manuals available for the design of waterway openings, (Reference 6 and 45).

Figure 1-3-1. Log Pearson Type III Exceedance Probability Plot

3.3.2.3 The Rational Method

3.3.2.3.1 General

a. The rational method is an old, simple, widely used (and often criticized) method employed in the determination of peak discharges from a given watershed. The method is based on the idea that the peak rate of surface outflow from a given watershed will be proportional to the watershed area and the average rainfall intensity over a period of time just sufficient for all parts of the watershed to contribute to the outflow (Reference 38). The constant of proportionality is then supposed to reflect all those characteristics of the watershed. In its simplest form, the rational formula is written as

Q = CiA **EQ 2**

where:

b. In general the rational method should be applied to drainage basins less than 200 acres in area and is best suited for well-defined drainage basins, such as urban areas.

3.3.2.3.2 Determining Runoff Coefficient

a. Values of runoff coefficient are given separately for rural areas and urban areas in Table 1-3-1 and Table 1-3-2, as taken from Schwab, et al (Reference 46).

Q = peak discharge (cubic feet per second – cfs)
C = ratio or peak runoff rate to average rainfall rate over the
time of concentration (runoff coefficient)
i = rainfall intensity (inches/hour)
A = area of watershed under consideration (acres)

Table 1-3-1. Rural Area Runoff Coefficient Values

Vegetation and
Topography
Soil Texture
Open Sandy
Loam
Clay and Silt
Loam Tight Clay
Woodland

Flat 0-5% slope 0.10 0.30 0.40
Rolling 5-10% slope 0.25 0.35 0.50

Hilly 10-30% slope 0.30 0.50 0.60

Pasture

Flat 0.10 0.30 0.40
Rolling 0.16 0.36 0.55
Hilly 0.22 0.42 0.60

Cultivated

Flat 0.30 0.50 0.60
Rolling 0.40 0.60 0.70
Hilly 0.52 0.72 0.82

Natural Waterways

b. The runoff coefficient is influenced by many variables, and does not remain constant during a given storm. Thus, "engineering judgment" must be liberally applied in the selection of the coefficient magnitude.

3.3.2.3.3 Determining Rainfall Intensity

a. The rainfall intensity is the average value in inches per hour during the time of concentration which, by definition, is the time required for runoff to flow from the most remote part of the drainage area to the outlet structure.

b. Rainfall intensity relations will usually fit the following equation:

EQ 3

where:

c. As an example, Fair et al (Reference 22) found that in Indiana the following magnitudes apply:

$5 < c < 50$ $0 < d < 30$ $0.1 < m < 0.5$ $0.4 < n < 1.0$

Table 1-3-2. Urban Area Runoff Coefficient Values

Description of Area Runoff Coefficients

Business

Downtown 0.70 to 0.95
Neighborhood 0.50 to 0.70

Residential

Single-family 0.30 to 0.50
Multi-units, detached 0.40 to 0.60
Multi-units, attached 0.60 to 0.75
Residential (suburban) 0.25 to 0.40
Apartment 0.50 to 0.70

Industrial

Light 0.50 to 0.80
Heavy 0.60 to 0.90

Miscellaneous

Parks, cemeteries 0. 10 to 0.25
Playgrounds 0.20 to 0.35
Railroad yard 0.20 to 0.35
Unimproved 0.10 to 0.30

i = intensity (inches/hour)
T = return period (years)
t = storm duration (minutes)
c, d, m, n = regional coefficients

$$i = \frac{cT^m}{(t + d)^n}$$

d. Reference may also be made to the rainfall intensity curves presented by various publications

(Reference 24 and 26). These data may be used in the rational formula in concert with the appropriate storm duration.

e. The appropriate return period has been discussed in Article 3.3.2.2 and the reader is referred to that discussion.

f. The appropriate storm duration may be calculated from the information found in Table 1-3-3.

Table 1-3-3. Storm Duration Calculations

Name Equation for t Notes

Ragan (1972)
(Reference 45)
n varies from about 0.025 to 0.040 for flow over natural earthen materials **EQ 4**

Kerby (1959)
(Reference 34)
L < 1200 ft; N as in Table 1-3-4 **EQ 5**

Izzard (1946)
(Reference 32)
for iL < 500; c as in Table 1-3-5 **EQ 6**

Table 1-3-4. Values of N in the Kerby Formula

Type of Surface	N
Smooth impervious surface	0.02
Smooth bare packed soil	0.10
Poor grass, cultivated row crops or moderately rough bare surface	0.20
Deciduous timberland	0.60
Pasture or average grass	0.40
Conifer timberland, deciduous timberland with deep forest litter or dense grass	0.80

Table 1-3-5. Values of c in Izzard Formula

Surface	N
Smooth asphalt surface	0.007
Concrete pavement	0.012
Tar and gravel pavement	0.017
Closely clipped sod	0.046
Dense bluegrass turf	0.060

t $L^{0.6}n^{0.6}$
$i^{0.4}S^{0.3}$
= ---------------------

t 0.827 NL
$s^{0.5}$
(---------)
( ) 0.467
=

t 2
60
------0.0007i + c
$S^{1/3}$
----------------------------LiL $^{-2/3}$
43200
= -----------------

t = overland flow time (min), considered to be equal to storm duration
L = basin length (ft)
S = basin slope (ft/ft)
i = rainfall intensity (in/hr)

3.3.2.3.4 Application of the Rational Method

a. Determine the basin area A (acres) by using USGS topographical maps, maps developed from a survey of the area, or plans made specifically for the basin. This area can then be found by use of a planimeter, counting squares, etc.

b. By the use of Tables for rural and urban areas from Article 3.3.2.3.2, find the appropriate value of C (runoff coefficient). If the land is a conglomerate of uses, a composite C value may be determined by:

EQ 7

where:

C_1, $C_2 \ldots C_n$ are the runoff coefficients associated with component areas A_1, $A_2 \ldots A_n$ and

$A_t = A_1 + A_2 + \ldots A_n$.

c. Determine the magnitude of rainfall intensity. The storm duration for the basin can be determined by using one of the equations listed in the Table 1-3-3 in Article 3.3.2.3.3. This magnitude is found by knowing the basin length, slope, and cover.

d. Determine rainfall intensity by using EQ 3 with appropriate coefficients, or by entering an intensityduration-

frequency diagram (Reference 24 and 26).

e. The data of paragraph a, paragraph b and paragraph d are then inserted into EQ 1 to yield the predicted peak discharge.

3.3.2.4 Soil Conservation Service Curve Number Method

3.3.2.4.1 The Theory

a. This method develops the quantity of runoff from a given amount of precipitation, and considers the effects of basin soil and cover types, rainfall depth, and antecedent moisture conditions (Reference 49).

b. The total runoff is calculated as the difference between total rainfall and total abstraction, which is the sum of total infiltration and how much water is used to initially wet the surface and fill surface depressions. The method assumes that the ratio of runoff to available water is the same as the ratio of infiltration to ultimate total abstraction. The resulting equation is:

EQ 8

where:

c. This calculation is made for 4 or 5 storms of different convenient storm durations to assess which produces the most severe condition. Often local custom dictates which storms must be examined.

R(t) = runoff (cumulative) (inches)

P(t) = total rainfall (inches)

S = ultimate total abstraction

C_{comp}

$(C_1A_1 + C_2A_2 + YC_nA_n)$

A_t

= --

$R(t)\ [P(t) - 0.2S]_2$

$[P(t) + 0.8S]$

= ------------------------------------

3.3.2.4.2 Determining the Parameter S

a. Soil type is variable, and four groupings have been created based on infiltration capacity. Table 1-3-6 lists these soil criteria:

b. Also to be considered is the antecedent moisture condition (AMC), which is an indication of how much rain has fallen on the basin recently. See Table 1-3-7.

c. Then, knowing the soil group and the antecedent moisture condition class, a curve number (CN) is established from Table 1-3-8 which is representative of a large variety of conditions.

Table 1-3-6. Soil Type Groupings

Soil

Group Characteristics

A	Soils in this category have a high infiltration rate even when thoroughly wetted and consist mainly of deep, well-to excessively-drained sands or gravels. (Low runoff potential)
B	Soils in this category have moderate infiltration rates when thoroughly wetted and consist of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse textures.
C	Soils in this category have slow infiltration rates when thoroughly wetted and consist mainly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine textures.
D	Soils in this category have a very slow infiltration rate when thoroughly wetted and consist mainly of clay soils with high swelling potential, soils with a permanently high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. (High runoff potential)

Table 1-3-7. Determining Antecedent Moisture Condition

Antecedent Moisture Condition Class	**5-Day Antecedent Rainfall (inches)**	
	Dormant Season	**Growing Season**
I	Less than 0.5	Less than 1.4
II	0.5 →1.1	1.4 → 2.1
III	over 1.1	over 2.1

Table 1-3-8. Curve Numbers for Various Cover and Soil Types [AMC = II] (Reference 50)

Land Use Description	**Hydrologic Soil Group**			
	A	**B**	**C**	**D**
Cultivated Land: without conservation treatment	72	81	88	91
with conservation treatment	62	71	78	81
Pasture or range land: poor condition	68	79	86	89
good condition	39	61	74	80
Meadow: good condition	30	58	71	78
Wood or forest land: thin stand, poor cover, no mulch	45	66	77	83
good cover	25	55	70	77

Open spaces: lawns, parks, golf course, cemeteries, etc.,
good condition: grass cover on 75% or more of the area
fair condition: grass cover on 50% to 75% of the area
39
49
61
69
74
79
80
84
Commercial and business areas (85% impervious) 89 92 94 95
Industrial districts (72% impervious) 81 88 91 93
Residential (Notes 1 and 4):
Average Lot Size Average % Impervious (Note 2)
1/8 acre or less 65
1/4 acre 38
1/3 acre 30
1/2 acre 25
1 acre 20
77
61
57
54
51
85
75
72
70
68
90
83
81
80
79
92
87
86
85
84
Paved parking lots, roofs, driveways, etc. (Note 3) 98 98 98 98
Streets and roads:
paved with curbs and storm sewers (Note 3)
gravel
dirt
98
76
72
98
85
82
98
89
87
98
91

89
Urban areas:
Low density (15-18% impervious surfaces)
Medium density (21-27% impervious surfaces)
High density (50-75% impervious surfaces)
69-71
71-73
73-75
75-78
77-80
79-82
82-84
84-86
86-88
86
88
90

Note 1: Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

Note 2: The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

Note 3: In some warmer climates of the country a curve number of 95 may be used.

Note 4: Curve numbers may vary with different parts of the country. The local SCD office should be contacted for recommended numbers in that locality.

Table 1-3-8. Curve Numbers for Various Cover and Soil Types [AMC = II] (Reference 50) (Continued)

Land Use Description
Hydrologic Soil Group
A B C D

d. Knowing the curve number (CN), the value of S (ultimate total abstraction) is found from:

EQ 9

e. If different antecedent moisture conditions exist than those of Group II, the conversions to CN found in Table 1-3-9 may be used (Reference 49):

Table 1-3-9. Antecedent Moisture Condition to Curve Number Conversion

CN for Condition II	CN for Condition I	CN for Condition III	CN for Condition II	CN for Condition I	CN for Condition III
100	100	100	61	41	78
99	97	100	60	40	78
98	94	99	59	39	77
97	71	99	58	38	76
96	89	99	57	37	75
95	87	98	56	36	75
94	85	98	55	35	74
93	83	98	54	34	73
92	81	97	53	33	72
91	80	97	52	32	71
90	78	96	51	31	70

89	76	96	50	31	70
88	75	95	49	30	69
87	73	95	48	29	68
86	72	94	47	28	67
85	70	94	46	27	66
84	68	93	45	26	65
83	67	93	44	25	64
82	66	92	43	25	63
81	64	92	42	24	62
80	63	91	41	23	61
79	62	91	40	22	60
78	60	90	39	21	59
77	59	89	38	21	58
76	58	89	37	20	57
75	57	88	36	19	56
74	55	88	35	18	55
73	54	87	34	18	54
72	53	86	33	17	53
71	52	86	32	16	52
70	51	85	31	16	51
69	50	84	30	15	50
68	48	84			
67	47	83	25	12	43
66	46	82	20	9	37
65	45	82	15	6	30
64	44	81	10	4	22
63	43	80	5	2	13
62	42	79	0	0	0

S 1000
CN
= ------------ – 10

3.3.2.4.3 Application of the CN Method – Total Flow

a. Let us assume the CN has been established, and S has been calculated. By selecting a rainfall depth for a particular storm, the runoff may be calculated using EQ 8. The runoff (inches) is multiplied by the basin area, and the result is converted to units of volume of runoff for the basin.

b. For a basin which has a conglomerate of soil types, a weighted CN may be calculated from:

EQ 10

where:

CN_1, CN_2…CN_n are the curve numbers associated with component areas A_1, A_2 … A_n and $A_t = A_1 + A_2 \ldots + A_n$.

c. The application yields the total amount of runoff from the given rainfall. For design, then, this is repeated for storms of different duration and amount of rainfall.

3.3.2.4.4 Determining Peak Flow from a Rainfall Event

a. The peak flow to be expected from a storm many times is more important than the total flow because the peak flow must be carried by the waterway opening. The following procedure is adapted from the principles of the unit hydrograph (Reference 49) and is intended for the designer's use as a prediction tool.

b. The steps in the procedure follow.

(1) For the basin, determine:
(a) the basin area (square miles)
(b) the basin *curve number* for the soil type(s) and antecedent moisture condition (as in Article 3.3.2.4.2)
(c) the depth, and time distribution of the rainfall for the storm in question.
(2) Find the time of concentration, t_c. This value may be obtained using Figure 1-3-2 for rural watersheds or Figure 1-3-3 for urbanized watersheds. In using Figure 1-3-2, the ordinate is entered with the travel path slope, then the diagonal line which represents the basin characteristics is intercepted, and a velocity is found by reading the abscissa. The time is then found by dividing the travel path by the velocity and appropriate conversions. Figure 1-3-3 utilized the curve number, travel length, and watershed in the calculation of the watershed lag L. This lag is converted to t_c by the empirical relationship $L = 0.6t_c$.
(3) Calculate peak flow:

CN_{comp}

$(CN_1A_1 + CN_2A_2 + YCN_nA_n)$

A_t

= ---

q_p

$484\ AR$

$0.667t_c$

= ---------------------

where:

c. Repeat paragraph b(1) through paragraph b(3) for storms of different duration and amount of rainfall; for example, durations of from 0.2 to 24 hours, or as load experience dictates, using associated rainfalls from hydrologic records for the locality. More detailed information can be found in Reference 49, as obtained from U.S. Supt. of Documents.

3.3.3 SUMMARY (1984)

The procedures of Article 3.3.2 will provide the design engineer with a good estimate of the capacity required for his waterway opening provided the results are used with engineering judgement. For those interested in additional tops in hydrology for engineering use, Reference 8, 17, and 35 are highly recommended.

q_p = peak flow (c.f.s.)
A = basin area (sq miles)
R = runoff depth (inches) (as calculated by EQ 8)
t_c = time of concentration (hours)

Figure 1-3-2. Velocities of Flow, Rural Watersheds (**Reference 49**)

Figure 1-3-3. Curve Number Method for Estimating Lag (L) [L = 0.6tc], Urban Watersheds (**Reference 49**)

SECTION 3.4 BASIC CONCEPTS AND DEFINITONS OF SCOUR[1]

3.4.1 SCOUR (2005)

3.4.1.1 Definition

Scour is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams and from around the piers and abutments of bridges. Different materials scour at different rates. Loose granular soils are rapidly eroded by flowing water, while cohesive or cemented soils are

more scour-resistant. However, ultimate scour in cohesive or cemented soils can be as deep as scour in sandbed
streams. Under constant flow conditions, scour will reach maximum depth in sand-bed and gravel-bed material in hours; cohesive bed material in days; glacial till, sandstones, and shale in months; limestone in years, and dense granite in centuries. Under flow conditions typical of actual bridge crossings, several floods may be needed to attain maximum scour.

Determining the magnitude of scour is complicated by the cyclic nature of the scour process. Scour can be deepest near the peak of a flood, but hardly visible as floodwaters recede and scour holes refill with sediment.

3.4.1.2 Clear-Water and Live-Bed Scour

There are two conditions for scour at a bridge: clear-water and live-bed scour. Clear-water scour occurs when there is no movement of the bed material in the flow upstream of the crossing or the bed material being transported in the upstream reach is transported in suspension through the scour hole at the pier or abutment
at less than the capacity of the flow. At the pier or abutment the acceleration of the flow and vortices created by
these obstructions cause the bed material around them to move. Live-bed scour occurs when there is transport
of bed material from the upstream reach into the crossing. Live-bed local scour is cyclic in nature; that is, the scour hole that develops during the rising stage of a flood refills during the falling stage.

Typical clear-water scour situations include (1) coarse-bed material streams, (2) flat gradient streams during low flow, (3) local deposits of larger bed materials that are larger than the biggest fraction being transported by
the flow (rock riprap is a special case of this situation), (4) armored streambeds where the only locations that tractive forces are adequate to penetrate the armor layer are at piers and/or abutments, and (5) vegetated channels or overbank areas.

During a flood event, bridges over streams with coarse-bed material are often subjected to clear-water scour at
low discharges, live-bed scour at the higher discharges and then clear-water scour at the lower discharges on the falling stages. Clear-water scour reaches its maximum over a longer period of time than live-bed scour (Figure 1-3-4). This is because clear-water scour occurs mainly in coarse-bed material streams. In fact, clearwater
scour may not reach a maximum until after several floods. Maximum clear-water pier scour is about 10 percent greater than the equilibrium local live-bed pier scour.

1 Sections 3.4 through 3.6 contain condensed material from Federal Highway Association (FHWA), U.S. Army Corps of Engineers (USACE) and American Society of Civil Engineers (ASCE) publications; primarily FHWA HEC-11, HEC-14, HEC-18, HEC-20, HEC-23, HDS-2 and HDS-6. For more detailed analysis and design information, refer to these organizations and publications. The bibliography at the end of each subsection contains additional references.

Live-bed pier scour in sand-bed streams with a dune bed configuration fluctuates about the equilibrium scour depth (Figure 1-3-4). This is due to the variability of the bed material sediment transport in the approach flow
when the bed configuration of the stream is dunes. However, with the exception of crossings over large rivers (i.e., the Mississippi, Columbia, etc.), the bed configuration in sand-bed streams will plane out during flood flows due to the increase in velocity and shear stress. For general practice, the maximum depth of pier scour is
approximately 10 percent greater than equilibrium scour. For a discussion of bedforms in alluvial channel flow,
see Chapter 3 of HDS 6.

3.4.2 LONG-TERM ELEVATION STREAMBED CHANGES (AGGRADATION AND

DEGRADATION) (2005)

Aggradation and degradation are long-term streambed elevation changes due to natural or man-induced causes
which can affect the reach of the river on which the bridge is located. Aggradation involves the deposition of material eroded from the channel or watershed upstream of the bridge; whereas, degradation involves the lowering or scouring of the streambed due to a deficit in sediment supply from upstream.

Long-term bed elevation changes may be the natural trend of the stream or the result of some modification to the stream or watershed. The streambed may be aggrading, degrading, or in relative equilibrium in the vicinity
of the bridge crossing. Long-term aggradation and degradation do not include the cutting and filling of the streambed in the vicinity of the bridge that might occur during a runoff event (contraction and local scour). A long-term trend may change during the life of the bridge.

These long-term changes are the result of modifications to the stream or watershed. Such changes may be the result of natural processes or human activities. The engineer must assess the present state of the stream and watershed and then evaluate potential future changes in the river system. From this assessment, the longterm streambed changes must be estimated.

Figure 1-3-4. Pier Scour Depth in a Sand-bed Stream as a Function of Time

3.4.3 CONTRACTION SCOUR (2005)

Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction of the stream channel or by a bridge. It also occurs when overbank flow is forced back to the channel by railway embankments at the approaches to a bridge. From continuity, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction and more bed material is removed from the contracted reach than
is transported into the reach. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases and, in the riverine situation, the velocity and shear stress decrease until relative equilibrium is reached; i.e., the quantity of bed material that is transported into the reach is equal to that removed from the reach, or the bed shear stress is decreased to a value such that no sediment is transported out of the reach. Contraction scour, in a natural channel or at a bridge crossing, involves removal of material from the bed across all or most of the channel width and can occur
as either clear-water or live-bed scour.

Live-bed contraction scour is typically cyclic; for example, the bed scours during the rising stage of a runoff event and fills on the falling stage. The cyclic nature of contraction scour causes difficulties in determining contraction scour depths after a flood. The contraction of flow at a bridge can be caused by either a natural decrease in flow area of the stream channel or by abutments projecting into the channel and/or piers blocking a
portion of the flow area. Contraction can also be caused by the approaches to a bridge cutting off floodplain flow. This can cause clear-water scour on a setback portion of a bridge section or a relief bridge because the floodplain flow does not normally transport significant concentrations of bed material sediments. This clearwater
picks up additional sediment from the bed upon reaching the bridge opening. In addition, local scour at abutments may well be greater due to the clear-water floodplain flow returning to the main channel at the end
of the abutment.

Other factors that can cause contraction scour are (1) natural stream constrictions, (2) long railroad approaches to the bridge over the floodplain, (3) ice formations or jams, (4) natural berms along the banks due
to sediment deposits, (5) debris, (6) vegetative growth in the channel or floodplain, and (7) pressure flow.

3.4.4 LOCAL SCOUR (2005)

Local scour involves removal of material from around piers, abutments, spurs, and embankments. Local scour can be either clear-water or live-bed scour. The basic mechanism causing local scour at piers or abutments is

the formation of vortices (known as the horseshoe vortex) at their base (Figure 1-3-5). The horseshoe vortex results from the pileup of water on the upstream surface of the obstruction and subsequent acceleration of the
flow around the nose of the pier or abutment. The action of the vortex removes bed material from around the base of the obstruction. The transport rate of sediment away from the base region is greater than the transport
rate into the region, and, consequently, a scour hole develops. As the depth of scour increases, the strength of the horseshoe vortex is reduced, thereby reducing the transport rate from the base region. Eventually, for livebed
local scour, equilibrium is reestablished between bed material inflow and outflow and scouring ceases. For clear-water scour, scouring ceases when the shear stress caused by the horseshoe vortex equals the critical shear stress of the sediment particles at the bottom of the scour hole.

In addition to the horseshoe vortex around the base of a pier, there are vertical vortices downstream of the pier
called the wake vortex (Figure 1-3-5). Both the horseshoe and wake vortices remove material from the pier base region. However, the intensity of wake vortices diminishes rapidly as the distance downstream of the pier
increases. Therefore, immediately downstream of a long pier there is often deposition of material.

Factors which affect the magnitude of local scour depth at piers and abutments are (1) velocity of the approach
flow, (2) depth of flow, (3) width of the pier, (4) discharge intercepted by the abutment and returned to the main
channel at the abutment (in laboratory flumes this discharge is a function of projected length of an abutment into the flow), (5) length of the pier if skewed to flow, (6) size and gradation of bed material, (7) angle of attack
of the approach flow to a pier or abutment, (8) shape of a pier or abutment, (9) bed configuration, and (10) ice
formation or jams and debris.

3.4.5 LATERAL STREAM MIGRATION (2005)

In addition to the types of scour mentioned above, naturally occurring lateral migration of the main channel of
a stream within a floodplain may affect the stability of piers in a floodplain, erode abutments or the approach embankments, or change the total scour by changing the flow angle of attack at piers and abutments. Factors that affect lateral stream movement also affect the stability of a bridge foundation.

Streams are dynamic. Areas of flow concentration continually shift banklines, and in meandering streams having an "S-shaped" planform, the channel moves both laterally and downstream. A braided stream has numerous channels which are continually changing. In a braided stream, the deepest natural scour occurs when two channels come together or when the flow comes together downstream of an island or bar. This scour
depth has been observed to be 1 to 2 times the average flow depth.

A bridge is static. It fixes the stream at one place in time and space. A meandering stream whose channel moves laterally and downstream into the bridge reach can erode the approach embankment and can affect contraction and local scour because of changes in flow direction. A braided stream can shift under a bridge and
have two channels come together at a pier or abutment, increasing scour.

Figure 1-3-5. Schematic Representation of Scour at a Cylindrical Pier

Factors that affect lateral shifting of a stream and the stability of a bridge are the geomorphology of the stream,
location of the crossing on the stream, flood characteristics, the characteristics of the bed and bank material, and wash load. It is difficult to anticipate when a change in planform may occur. It may be gradual or the result of a single major flood event. Also, the direction and magnitude of the movement of the stream are not easily predicted. While it is difficult to evaluate the vulnerability of a bridge due to changes in planform, it is important to incorporate potential planform changes into the design of new bridges and the design of countermeasures for existing bridges.
Countermeasures for lateral shifting and instability of the stream may include changes in the bridge design, construction of river control works, protection of abutments with riprap, or careful monitoring of the river in a
bridge inspection program. **Serious consideration should be given to placing footings/foundations located on floodplains at elevations the same as those located in the main channel**. Control of lateral shifting requires river training works, bank stabilizing by riprap, and/or guide banks.

3.4.6 TOTAL SCOUR (2005)

Total scour at a railroad crossing is comprised of three components:
(1) Long-term degradation of the river bed
(2) Contraction scour at the bridge
(3) Local scour at the piers or abutments
These three scour components are added to obtain the total scour at a pier or abutment. This assumes that each component occurs independent of the other. Considering the components additive adds some conservatism to the design. In addition, **lateral migration** of the stream must be assessed when evaluating total scour at bridge piers and abutments.

3.4.7 REFERENCES FOR SECTION 3.4 (2005)

Richardson, E.V. and Davis, S.R., 2001. "Evaluating Scour at Bridges," Fourth Edition, Report FHWA NHI 01-001, Federal Highway Administration, Hydraulic Engineering Circular No. 18, U.S. Department of Transportation, Washington, D.C.
Richardson, E.V., Simons, D.B., and Lagasse, P.F., 2001. "River Engineering for Highway Encroachments – Highways in the River Environment," Report FHWA NHI 01-004, Federal Highway Administration, Hydraulic Design Series No. 6, Washington, D.C.

SECTION 3.5 CALCULATING SCOUR

3.5.1 PREDICTING AGGRADATION AND DEGRADATION (2005)

3.5.1.1 Long-Term Bed Elevation Changes

Long-term bed elevation changes may be the natural trend of the stream or may be the result of some modification to the stream or watershed. The streambed may be aggrading, degrading, or in relative equilibrium in the vicinity of the bridge crossing. The problem for the engineer is to estimate the long-term bed
elevation changes that will occur during the life of the structure.

Factors that affect long-term bed elevation changes are dams and reservoirs (up- or downstream of the bridge),
changes in watershed land use (urbanization, deforestation, etc.), channelization, cutoffs of meander bends (natural or man-made), changes in the downstream channel base level (control), gravel mining from the streambed, diversion of water into or out of the stream, natural lowering of the fluvial system, movement of a bend and bridge location with respect to stream planform, and stream movement in relation to the crossing. Tidal ebb and flood may degrade a coastal stream; whereas, littoral drift may result in aggradation. The

elevation of the bed under bridges over a tributary to a larger stream will follow the trend of the larger stream
unless there are controls. Controls could be bedrock, dams, culverts or other structures. The changes in bed elevation decrease the further upstream the bridge is from the confluence with another stream or from other bed elevation controls.

The U.S. Army Corps of Engineers (USACE), U.S. Geological Survey (USGS), and other Federal and State agencies should be contacted concerning documented long-term streambed variations. If no data exist or if such data require further evaluation, an assessment of long-term streambed elevation changes for riverine streams should be made using the principles of river mechanics. Such an assessment requires the consideration of all influences upon the bridge crossing, i.e., runoff from the watershed to a stream (hydrology),
sediment delivery to the channel (watershed erosion), sediment transport capacity of a stream (hydraulics), and
response of a stream to these factors (geomorphology and river mechanics). Significant morphologic impacts can result from human activities. The assessment of the impact of human activities requires a study of the history of the river, as well as a study of present water and land use and stream control activities.

3.5.1.2 Estimating Long-Term Aggradation and Degradation

The following sections outline procedures that can assist in identifying long-term trends in vertical stability.

Bridge Inspection Records

The bridge inspection reports for railroad or highway bridges on the stream where a new or replacement bridge
is being designed are an excellent source of data on long-term aggradation or degradation trends. Also, inspection reports for bridges crossing streams in the same area or region should be studied. Railroad bridges
sometimes have records with a long history going back 100 years or more that document streambed conditions
during original construction. For most highway bridges, the biannual inspection includes taking the elevation and/or cross section of the streambed under the bridge. These elevations are usually referenced to the bridge,
but these relative bed elevations will show trends and can be referenced to sea level elevations. Successive cross sections from a series of bridges in a stream reach can be used to construct longitudinal streambed profiles through the reach.

Gaging Station Records

The USGS and many State Water Resource and Environmental agencies maintain gaging stations to measure stream flow. In the process they maintain records from which the aggradation or degradation of the streambed
can be determined. Gaging station records at the bridge site, on the stream to be bridged and in the area or region can be used.

Where an extended historical record is available, one approach to using gaging station records to determine long-term bed elevation change is to plot the change in stage through time for a selected discharge. This approach is often referred to as establishing a "specific gage" record.

Figure 1-3-6 shows a plot of specific gage data for a discharge of 500 cfs from about 1910 to 1980 for Cache Creek in California. Cache Creek has experienced significant gravel mining with records of gravel extraction quantities available since about 1940. When the historical record of cumulative gravel mining is compared to the specific gage plot, the potential impacts are apparent. The specific gage record shows more than 10 ft of long-term degradation in a 70-year period.

Geology and Stream Geomorphology

The geology and geomorphology of the site needs to be studied to determine the potential for long-term bed elevation changes at the bridge site. Quantitative techniques for streambed aggradation and degradation analyses are covered in detail in HEC-20. These techniques include:

- Incipient motion analysis
- Analysis of armoring potential

• Equilibrium slope analysis
• Sediment continuity analysis
Sediment transport concepts and equations are discussed in detail in HDS 6.

Figure 1-3-6. Specific Gage Data for Cache Creek, California

Computer Models

Sediment transport computer models can be used to determine long-term aggradation or degradation trends. These computer models route sediment down a channel and adjust the channel geometry to reflect imbalances
in sediment supply and transport capacity. The BRI-STARS and HEC-6 models are examples of sediment transport models that can be used for single event or long-term estimates of changes in bed elevation. The information needed to run these models includes:

• Channel and floodplain geometry
• Structure geometry
• Roughness
• Geologic or structural vertical controls
• Downstream water surface relationship
• Event or long-term inflow hydrographs
• Tributary inflow hydrographs
• Bed material gradations
• Upstream sediment supply
• Tributary sediment supply
• Selection of appropriate sediment transport relationship
• Depth of alluvium

These models perform hydraulic and sediment transport computations on a cross section basis and adjust the channel geometry prior to proceeding with the next time step. The actual flow hydrograph can be used as input.

Aggradation, Degradation, and Total Scour

Using all the information available estimate the long-term bed elevation change at the bridge site for the design
life of the bridge. Usually, the design life is 100 years. **If the estimate indicates that the stream will degrade, use the elevation after degradation as the base elevation for contraction and local scour. That is, total scour must include the estimated long-term degradation.** If the estimate indicates that the stream will aggrade, then (1) make note of this fact to inspection and maintenance personnel, and (2) use existing ground elevation as the base for contraction and local scour.

3.5.2 PREDICTING LATERAL MIGRATION (2005)

3.5.2.1 Initiation of Meanders

Although there is no completely satisfactory explanation of how or why meanders develop, it is known that meanders are initiated by localized bank retreat which alternates from one side of the channel to the other in a
more or less regular pattern. The primary features of the flow pattern through meander bends are:

• Superelevation of the water surface against the outside (convex) bank (Figure 1-3-7A)
• Transverse current directed towards the outer bank at the surface and towards the inner bank at the bed producing a secondary circulation additional to the main downstream flow (Figure 1-3-7B)
• Maximum-velocity current which moves from near the inner bank at the bend entrance to near the outer bank at the bend exit, crossing the channel at the zone of maximum bend curvature

The transverse current and the primary downstream flow component combine to produce the helicoidal motion
to the flow. The superelevation of the water surface against the outer bank of a bend produces a locally steep downstream energy gradient and, in turn, a zone of maximum boundary shear stress (τ_b) in close proximity to
the outer bank just downstream of the bend apex (Figure 1-3-7A).
Secondary currents, which are usually weaker than primary ones, influence the distribution of velocity and boundary shear stress. The bend cross-section can be divided into three regions relative to the pattern of secondary flow (Figure 1-3-7B):

- Mid-channel region, helicoidal flow is well established passing nearly 90 percent of the flow
- Cell of opposite circulation develops in the *outer bank region*: the strength of this cell increases with discharge, the steepness of the bar, and the acuteness of the bend
- Inner bank region where shoaling over the point bar induces a net outward flow, forcing the core of maximum velocity more rapidly toward the outer bank; increasing stage tends to reduce the shoaling, allowing an inward component of near-bed flow over the bar top

The location and timing of the flow pattern varies with discharge, bend tightness, and cross-sectional form. Primary currents are dominant at high discharges because the main flow follows a straighter path, but secondary currents are relatively strong at intermediate discharges.
The pattern of primary and secondary currents influences the distribution of erosion and deposition in meanders. In general, erosion in the bend is concentrated along the outer bank downstream of the bend apex where the currents are strongest, while point bar building predominates in a parallel position along the opposite bank, with material supplied by longitudinal and transverse currents. This produces a largely downvalley component to meander migration.

3.5.2.2 Evaluation and Prediction of Lateral Migration

In general, most streams are sinuous to some degree and the majority of bank retreat and lateral migration occurs along meander bends. As such, the following discussion on evaluating and predicting lateral migration will focus on meander bends. One of the most practical methods for determining lateral stability and migration
rates involves the analysis of sequential historic aerial photographs, maps, and surveys (Lagasse et al. 2003).

The most accurate means of measuring changes in channel geometry and lateral adjustments is through repetitive surveys of the channel cross section. However, this data is rarely available. The next easiest and relatively accurate method of determining migration rates and direction is through the comparison of sequential historical aerial photography (photos), maps, and surveys. Accuracy in such an analysis is greatly dependent on the period over which
migration is evaluated, the amount and magnitude of internal and external perturbations forced on the system over time, and the number and quality of sequential aerial photos and maps. The analysis will be much more accurate for a
channel that has coverage consisting of multiple data sets (aerial photos, maps, and surveys) covering a long period of
time (several tens of years to more than 100 years) versus an analysis consisting of only two or three data sets covering
a short time period (several years to a few tens of years). Predictions of migration for channels that have been extensively modified or have undergone major adjustments attributable to extensive land use changes will be much less
reliable than those made for channels in relatively stable watersheds.
Historical aerial photos and maps can be obtained from a number of federal, state, and local agencies (Lagasse et al. 2003). Extensive topographic map coverage of the United States at a variety of scales can be obtained from the local or
regional offices of the U.S. Geological Survey. In general, both air photos and maps are required to perform a

comprehensive and relatively accurate meander migration assessment. Since the scale of aerial photography is often approximate, contemporary maps are usually needed to accurately determine the true scale of air photos. Distortion of the image on aerial photos is also a common problem and becomes greater as one moves further away from the center of the photo. Expensive equipment, which is generally needed to rectify and eliminate aerial photo distortion, is often unavailable, so distortion and scale differences must be accounted for by some other means. The scale problem is easily rectified through the use of multiple distance measurements taken between common reference points on the photos and maps. The measurements of distance between several reference-point pairs common to both the photos and maps are then averaged to define an average scale for the photos. Common reference points can include cultural features such as building corners, roads or fences and their intersections, irrigation channels and canals, or natural features such as isolated rock outcrops, large boulders, and trees, drainages and stream confluences, and the irregular boundaries of water bodies.

Figure 1-3-7. Flow Patterns in Meanders

(A) Location of maximum boundary shear stress (τ_b), in a bend with a well-developed point bar (B) Secondary flow at a bend apex showing the outer bank cell and the shoaling-induced outward flow over the point bar.

The accurate delineation of a bankline on aerial photos is primarily dependent on the density of vegetation at the top of the bank. Top bank is easily defined if stereo-pairs of photos are available. However, single photos can be used relatively easily if one knows what to look for. For banks with little or no vegetation, the top of the bank is easily identified. The abrupt change between the water and the top of the bank along the convex bank in a bend or an eroding cutbank is defined by an abrupt change in the contrast and color (color photo) or gray tone (black and white photo).

Usually the water is significantly darker than the top of bank. Along the concave or inner bank of a bend, exposed bar sediment is lighter colored than the river or the top of bank. The top bank along a point bar is usually defined by persistent vegetation such as mature trees and shrubs.

Where vegetation becomes increasingly dense along a bank, small sections of the top of the bank may be visible such that a line can be drawn connecting the sections. Often, the top of the bank may be completely obscured by vegetation and one may be required to locate the top of the bank by approximation. In this case, one can assume that the trunks of the largest trees growing along the river are nearly vertical and are located just landward of the top of the bank. Therefore, a line that approximates the top of the bank may be drawn just riverward of the center of the tree. The amount of error involved with this method increases with decreasing stream size.

If the density of vegetation along a stream is such that an accurate delineation of the top of the bank cannot be made, then the use of the channel centerline may be required. The centerline is drawn with reference to bankfull conditions.

Therefore, the channel centerline can and often does cross the exposed portions of point bars. Usually the channel centerline can be delineated more easily than a bankline masked by vegetation since the centerline can be drawn equidistant from the edge of mature vegetation on either side of the channel.

There are three general methods of assessing lateral bank erosion and meander migration using maps and aerial photographs. The following discussion will deal with assessments using air photos, but the same methods can be used when making assessments or measurements from maps. In order of increasing complexity and accuracy, distances of lateral retreat can be:

• Estimated by visual comparison of two air photos flown at different times
• Measured by scaling distances directly from the bank to fixed reference points common to both photographs
• Measured on a drawing on which historic channel banklines taken from sequential air photos are superimposed at the same scale

The first method provides a preliminary assessment of stability, especially where significant changes in bank position have occurred. The second method requires measurements made along a line described by two reference points on either side of the bank that are common to both photos. The second method will usually only provide a few accurate measurements along the bank, depending on the number of reference points and the number of lines that can be drawn across the bend. Additional problems may be associated with the location of the lines since they may not be perpendicular to bank retreat nor allow a measurement at the point of maximum retreat.

The third method is relatively easy and accurate. This method requires that the banklines and the common reference points from each historic air photo be traced onto a transparent or semi-transparent sheet after they have been enlarged or reduced to a common scale. The channel centerline can also be delineated on the same sheet at the same time. Then, each bankline or centerline is transferred to and superimposed on a common sheet such that a sequential comparison of the banklines or centerlines can be made. The total bankline area eroded can be measured for each period and divided by the bankline length to define the average bank retreat. Dividing either the maximum distance or the average distance of bank retreat by the number of years between air photos results in a maximum or average migration rate, respectively. Drawing a line perpendicular to centerline at the location of maximum retreat defines the direction of maximum retreat. This process is repeated for each series of sequential photos. Based on the measurements between years, one may be able to define migration rates relative to significant hydrologic or geomorphic events. Overall rates can also be determined by summing the distances and dividing by the total number of years between the earliest and latest photos.

3.5.3 ESTIMATING CONTRACTION SCOUR (2005)

3.5.3.1 Contraction Scour Conditions

Contraction scour equations are based on the principle of conservation of sediment transport (continuity). In the case of live-bed scour, the fully developed scour in the bridge cross section reaches equilibrium when sediment transported into the contracted section equals sediment transported out. As scour develops, the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity.

For live-bed scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance. For clear-water scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material in the section. Normally, for both livebed and clear-water scour the width of the contracted section is constrained and depth increases until the limiting conditions are reached.

Live-bed contraction scour occurs at a bridge when there is transport of bed material in the upstream reach

into the bridge cross section. With live-bed contraction scour the area of the contracted section increases until,
in the limit, the transport of sediment out of the contracted section equals the sediment transported in.
Clear-water contraction scour occurs when (1) there is no bed material transport from the upstream reach into the downstream reach, or (2) the material being transported in the upstream reach is transported through
the downstream reach mostly in suspension and at less than capacity of the flow. With clear-water contraction
scour the area of the contracted section increases until, in the limit, the velocity of the flow (V) or the shear stress (J_o) on the bed is equal to the critical velocity (V_c) or the critical shear stress (J_c) of a certain particle size
(D) in the bed material.
There are four conditions (cases) of contraction scour at bridge sites depending on the type of contraction, and
whether there is overbank flow or relief bridges. Regardless of the case, contraction scour can be evaluated using two basic equations: (1) **live-bed** scour, and (2) **clear-water** scour. For any case or condition, it is only necessary to determine if the flow in the main channel or overbank area upstream of the bridge, or approaching
a relief bridge, is transporting bed material (live-bed) or is not (clear-water), and then apply the appropriate equation with the variables defined according to the location of contraction scour (channel or overbank).
To determine if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c of the D_{50} size of the bed material being considered for movement and compare it with
the mean velocity V of the flow in the main channel or overbank area upstream of the bridge opening. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V$), then clear-water contraction scour
will exist. If the critical velocity is less than the mean velocity ($V_c < V$), then live-bed contraction scour will exist. To calculate the critical velocity use the following equation:

$V_c = K_u y^{1/6} D^{1/3}$ **EQ 11**

where:
V_c = Critical velocity above which bed material of size D and smaller will be transported, m/s (ft/s)
y = Average depth of flow upstream of the bridge, m (ft)
D = Particle size for V_c, m (ft)
D_{50} = Particle size in a mixture of which 50 percent are smaller, m (ft)
K_u = 6.19 SI units
K_u = 11.17 English units

The D_{50} is taken as an average of the bed material size in the reach of the stream upstream of the bridge. It is a characteristic size of the material that will be transported by the stream. Normally this would be the bed material size in the upper 1 ft of the stream bed.
Live-bed contraction scour depths may be limited by armoring of the bed by large sediment particles in the bed material or by sediment transport of the bed material into the bridge crosssection.
Under these conditions, live-bed contraction scour at a bridge can be determined by calculating the scour depths using both the clear-water and live-bed contraction scour equations and using the smaller of the two depths.

3.5.3.2 Contraction Scour Cases

Four conditions (cases) of contraction scour are commonly encountered:
Case 1. Involves overbank flow on a floodplain being forced back to the main channel by the approaches to the bridge. Case 1 conditions include:
a. The river channel width becomes narrower either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river (Figure 1-3-8);
b. No contraction of the main channel, but the overbank flow area is completely obstructed by an embankment (Figure 1-3-9); or

c. Abutments are set back from the stream channel (Figure 1-3-10).

Case 2. Flow is confined to the main channel (i.e., there is no overbank flow). The normal river channel width becomes narrower due to the bridge itself or the bridge site is located at a narrower reach of the river (Figure 1-3-11 and Figure 1-3-12).

Case 3. A relief bridge in the overbank area with little or no bed material transport in the overbank area (i.e., clear-water scour) (Figure 1-3-13).

Case 4. A relief bridge over a secondary stream in the overbank area with bed material transport (similar to Case 1) (Figure 1-3-14).

Notes:

(1) **Cases 1, 2, and 4** may either be live-bed or clear-water scour depending on whether there is bed material transport from the upstream reach into the bridge reach during flood flows. To determine if there is bed material transport compute the critical velocity at the approach section for the D_{50} of the bed material using the equation given above and compare to the mean velocity at the approach section. To determine if the bed material will be washed through the contraction determine the ratio of the shear velocity (V^*) in the contracted section to the fall velocity (T) of the D_{50} of the bed material being transported from the upstream reach (see the definition of V^* in the live-bed contraction scour equation). If the ratio is much larger than 2, then the bed material from the upstream reach will be mostly suspended bed material discharge and may wash through the contracted reach (clear-water scour).

(2) **Case 1c is very complex.** The depth of contraction scour depends on factors such as (1) how far back from the bank line the abutment is set, (2) the condition of the overbank (is it easily eroded, are there trees on the bank, is it a high bank, etc.), (3) whether the stream is narrower or wider at the bridge than at the upstream section, (4) the magnitude of the overbank flow that is returned to the bridge opening, and (5) the distribution of the flow in the bridge section, and (6) other factors.

The main channel under the bridge may be live-bed scour; whereas, the set-back overbank area may be clear-water scour.

WSPRO or HEC-RAS can be used to determine the distribution of flow between the main channel and the set-back overbank areas in the contracted bridge opening. However, the distribution of flow needs to be done with care. Studies by Chang and Sturm (HEC-18) have shown that conveyance calculations do not properly account for the flow distribution under the bridge.

Figure 1-3-8. Case 1A: Abutments Project into Channel

Figure 1-3-9. Case 1B: Abutments at Edge of Channel

Figure 1-3-10. Case 1C: Abutments Set Back from Channel

Figure 1-3-11. Case 2A: River Narrows

Figure 1-3-12. Case 2B: Bridge Abutments and/or Piers Constrict Flow

Figure 1-3-13. Case 3: Relief Bridge Over Floodplain

Figure 1-3-14. Case 4: Relief Bridge Over Secondary Stream

If the abutment is set back only a small distance from the bank (less than 3 to 5 times the average depth of flow through the bridge), there is the possibility that the combination of contraction scour and abutment scour may destroy the bank. Also, the two scour mechanisms are not independent. Consideration should be given to using a guide bank and/or protecting the bank and bed under the bridge in the overflow area with rock riprap. See Section 3.6 for guidance on designing rock riprap.

(3) **Case 3** may be clear-water scour even though the floodplain bed material is composed of sediments with a critical velocity that is less than the flow velocity in the overbank area. The reasons for this are (1) there may be vegetation growing part of the year, and (2) if the bed material is fine sediments, the bed material discharge may go into suspension (wash load) at the bridge and not influence contraction scour.

(4) **Case 4** is similar to Case 3, but there is sediment transport into the relief bridge opening (live-bed scour). This case can occur when a relief bridge is over a secondary channel on the floodplain. Hydraulically this is no different from Case 1, but analysis is required to determine the floodplain discharge associated with the relief opening and the flow distribution going to and through the relief bridge. This information could be obtained from WSPRO or HEC-RAS.

3.5.3.3 Live-Bed Contraction Scour

A modified version of Laursen's 1960 equation for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section. The equation assumes that bed material is being transported
from the upstream section.

$y_s = y_2 - y_o$ = (average contraction scour depth) **EQ 13**

where:

y_1 = Average depth in the upstream main channel, m (ft)
y_2 = Average depth in the contracted section,m (ft)
y_o = Existing depth in the contracted section before scour, m (ft) (see Note 7)
Q_1 = Flow in the upstream channel transporting sediment, m^3/s (ft^3/s)
Q_2 = Flow in the contracted channel, m^3/s (ft^3/s)
W_1 = Bottom width of the upstream main channel that is transporting bed material, m (ft)
W_2 = Bottom width of the main channel in the contracted section less pier width(s), m (ft)
k_1 = Exponent determined below

V_*/ω	k_1	Mode of Bed Material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

y
y

$$= \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \qquad \textbf{EQ 12}$$

ω

Notes:

(1) Q_2 may be the total flow going through the bridge opening as in cases 1a and 1b. **It is not the total flow for Case 1c**. For Case 1c contraction scour must be computed separately for the main channel and the left and/or right overbank areas.

(2) Q_1 is the flow in the main channel upstream of the bridge, not including overbank flows.

(3) W_1 and W_2 are not always easily defined. In some cases, it is acceptable to use the topwidth of the main channel to define these widths. Whether topwidth or bottom width is used, it is important to be consistent so that W_1 and W_2 refer to either bottom widths or top widths.

(4) The average width of the bridge opening (W_2) is normally taken as the bottom width, with the width of the piers subtracted.

$V_* = (\tau_o/\rho)^{1/2} = (gy_1S_1)^{1/2}$, shear velocity in the upstream section, m/s (ft/s)

ϖ = Fall velocity of bed material based on the D_{50}, m/s (Figure 1-3-15)

g = Acceleration of gravity (9.81 m/s^2) (32.2 ft/s^2)

S_1 = Slope of energy grade line of main channel, m/m (ft/ft)

τ_o = Shear stress on the bed, Pa (N/m^2) (lb/ft^2)

ρ = Density of water (1000 kg/m^3) (1.94 slugs/ft^3)

Figure 1-3-15. Fall Velocity of Sand-sized Particles with Specific Gravity of 2.65 in Metric Units

(5) Laursen's equation will overestimate the depth of scour at the bridge if the bridge is located at the

upstream end of a natural contraction or if the contraction is the result of the bridge abutments and piers. At this time, however, it is the best equation available.

(6) In sand channel streams where the contraction scour hole is filled in on the falling stage, the y_0 depth may be approximated by y_1. Sketches or surveys through the bridge can help in determining the existing bed elevation.

(7) Scour depths with live-bed contraction scour may be limited by coarse sediments in the bed material armoring the bed. Where coarse sediments are present, it is recommended that scour depths be calculated for live-bed scour conditions using the clear-water scour equation (given in the next section) in addition to the live-bed equation, and that the smaller calculated scour depth be used.

3.5.3.4 Clear-Water Contraction Scour

The recommended clear-water contraction scour equation is:

$y_s = y_2 - y_o$ = (average contraction scour depth) **EQ 15**

where:

EQ 14 is a rearranged version of EQ 11.

Because D_{50} is not the largest particle in the bed material, the scoured section can be slightly armored. Therefore, the D_m is assumed to be 1.25 D_{50}. For stratified bed material the depth of scour can be determined by using the clear-water scour equation sequentially with successive D_m of the bed material layers.

3.5.3.5 Contraction Scour With Backwater

The live-bed contraction scour equation is derived assuming a uniform reach upstream and a long contraction

into a uniform reach downstream of the bridge. With live-bed scour the equation computes a depth after the

y_2 = Average equilibrium depth in the contracted section after contraction scour, m (ft)

y_o = Average existing depth in the contracted section, m (ft)

Q = Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W, m^3/s (ft^3/s)

D_m = Diameter of the smallest nontransportable particle in the bed material (1.25 D_{50}) in the contracted section, m (ft)

D_{50} = Median diameter of bed material, m (ft)

W = Bottom width of the contracted section less pier widths, m (ft)

K_u = 0.025 SI units

K_u= 0.0077 English units

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2}\right]^{3/7}$$ **EQ 14**

long contraction where the sediment transport into the downstream reach is equal to the sediment transport out. The clear-water contraction scour equations are derived assuming that the depth at the bridge increases until the shear-stress and velocity are decreased so that there is no longer any sediment transport. With the clear-water equations it is assumed that flow goes from one uniform flow condition to another. Both equations
calculate contraction scour depth assuming a level water surface ($y_s = y_2 - y_o$). A more consistent computation would be to write an energy balance before and after the scour. For live-bed the energy balance would be between the approach section (1) and the contracted section (2). Whereas, for clear-water scour it would be the
energy at the same section before (1) and after (2) the contraction scour.

Backwater, in extreme cases, can decrease the velocity, shear stress and the sediment transport in the upstream section. This will increase the scour at the contracted section. The backwater can, by storing sediment in the upstream section, change live-bed scour to clear-water scour.

3.5.4 ESTIMATING LOCAL PIER SCOUR (2005)

3.5.4.1 General

Local scour at piers is a function of bed material characteristics, bed configuration, flow characteristics, fluid properties, and the geometry of the pier and footing. The bed material characteristics are granular or non granular, cohesive or noncohesive, erodible or non erodible rock. Granular bed material ranges in size from silt
to large boulders and is characterized by the D_{50} and a coarse size such as the D_{84} or D_{90} size. Cohesive bed material is composed of silt and clay, possibly with some sand which is bonded chemically. Rock may be solid, massive, or fractured. It may be sedimentary or igneous and erodible or non erodible.

Flow characteristics of interest for local pier scour are the velocity and depth just upstream of the pier, the angle the velocity vector makes to the pier (angle of attack), and free surface or pressure flow. Fluid properties
are viscosity, and surface tension which for the field case can be ignored.

Pier geometry characteristics are its type, dimensions, and shape. Types of piers include single column, multiple columns, or rectangular; with or without friction or tip bearing piles; with or without a footing or pile
cap; footing or pile cap in the bed, on the surface of the bed, in the flow or under the deck out of the flow.

Important dimensions are the diameter for circular piers or columns, spacing for multiple columns, and width and length for solid piers. Shapes include round, square or sharp nose, circular cylinder, group of cylinders, or rectangular. In addition, piers may be simple or complex. A simple pier is a single shaft, column or multiple columns exposed to the flow. Whereas, a complex pier may have the pier, footing or pile cap, and piles exposed
to the flow.

Local scour at piers has been studied extensively in the laboratory; however, there is limited field data. The laboratory studies have been mostly of simple piers, but there have been some laboratory studies of complex piers. Often the studies of complex piers are model studies of actual or proposed pier configurations. As a result
of the many laboratory studies, there are numerous pier scour equations. In general, the equations are for livebed
scour in cohesionless sand-bed streams.

A graphical comparison of the more common equations is given in Figure 1-3-16. Some of the equations have velocity as a variable, normally in the form of a Froude Number. However, some equations, such as Laursen's do not include velocity. A Froude Number of 0.3 was used in Figure 1-3-16 for purposes of comparing commonly used scour equations. Based on a comparison of the equations with the laboratory data and available field data, the Colorado State University (CSU) equation is recommended to estimate pier scour.

3.5.4.2 Local Pier Scour Equation

To determine pier scour, an equation based on the CSU equation is recommended for both live-bed and clearwater
pier scour. The equation predicts maximum pier scour depths. The equation is:

As a Rule of Thumb, the maximum scour depth for round nose piers aligned with the flow is:

y_s = 2.4 times the pier width (a) for $Fr \leq 0.8$ **EQ 17**

y_s = 3.0 times the pier width (a) for $Fr > 0.8$

In terms of y_s/a, EQ 16 is:

where:

y_s = Scour depth, m (ft)

y_1 = Flow depth directly upstream of the pier, m (ft)

K_1 = Correction factor for pier nose shape from Figure 1-3-17 and Table 1-3-10

K_2 = Correction factor for angle of attack of flow from Table 1-3-11 or EQ 19

K_3 = Correction factor for bed condition from Table 1-3-12

a = Pier width, m (ft)

$$\frac{y_s}{y_1} = 2.0\, K_1 K_2 K_3 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43} \qquad \textbf{EQ 16}$$

Figure 1-3-16. Comparison of Scour Equations for Variable Depth Ratios (y/a) (HEC-18)

$$\frac{y_s}{a} = 2.0\, K_1 K_2 K_3 \left(\frac{y_1}{a}\right)^{0.35} Fr_1^{0.43} \qquad \textbf{EQ 18}$$

The correction factor, K_2, for angle of attack of the flow, θ, is calculated using the following equation:

If L/a is larger than 12, use L/a = 12 as a maximum in EQ 19 and Table 1-3-11. Table 1-3-11 illustrates the magnitude of the effect of the angle of attack on local pier scour.

Table 1-3-10. Correction Factor, K_1, for Pier Nose Shape

L = Length of pier, m (ft)

Fr_1 = Froude Number directly upstream of the pier = $V_1/(gy_1)^{1/2}$

V_1 = Mean velocity of flow directly upstream of the pier, m/s (ft/s)

g = Acceleration of gravity (9.81 m/s^2) (32.2 ft/s^2)

Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group of cylinders	1.0
(e) Sharp nose	0.9

$$K_2 = [\,Cos\,\theta + (L/a)Sin\,\theta\,]^{0.65} \quad \textbf{EQ 19}$$

Figure 1-3-17. Common Pier Shapes

Table 1-3-11. Correction Factor, K_2, for Angle of Attack, θ, of the Flow

Table 1-3-12. Increase in Equilibrium Pier Scour Depths, K_3, for Bed Condition

Notes:

(1) The correction factor K_1 for pier nose shape should be determined using Table 1-3-10 for angles of attack up to 5 degrees. **For greater angles, K_2 dominates and K_1 should be considered as 1.0**. If L/a is larger than 12, use the values for L/a = 12 as a maximum in Table 1-3-11 and EQ 19.

(2) The values of the correction factor K_2 should be applied only when the field conditions are such that the entire length of the pier is subjected to the angle of attack of the flow. Use of this factor will result in a significant over-prediction of scour if (1) a portion of the pier is shielded from the direct impingement of the flow by an abutment or another pier; or (2) an abutment or another pier redirects the flow in a direction parallel to the pier. For such cases, judgment must be exercised to reduce the value of the K_2 factor by selecting the effective length of the pier actually subjected to the angle of attack of the flow. EQ 19 should be used for evaluation and design. Table 1-3-11 is intended to illustrate the importance of angle of attack in pier scour computations and to establish a cutoff point for K_2 (i.e., a maximum value of 5.0).

(3) The correction factor K_3 results from the fact that for plane-bed conditions, which is typical of most bridge sites for the flood frequencies employed in scour design, the maximum scour may be 10 percent greater than computed with Equation 3.6. In the **unusual** situation where a dune bed configuration **with large dunes** exists at a site during flood flow, the maximum pier scour may be 30 percent greater than the predicted equation value. This may occur on very large rivers, such as the Mississippi. For smaller streams that have a dune bed configuration at flood flow, the dunes will be smaller and the maximum scour may be only 10 to 20 percent larger than equilibrium scour. For antidune bed configuration the maximum scour depth may be 10 percent greater than the computed equilibrium pier scour depth (see HDS 6).

(4) Piers set close to abutments (for example at the toe of a spill through abutment) must be carefully evaluated for the angle of attack and velocity of the flow coming around the abutment.

Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Angle = skew angle of flow

L = length of pier
Bed Condition Dune Height m K_3
Clear-Water Scour N/A 1.1
Plane bed and Antidune flow N/A 1.1
Small Dunes 3 > H > 0.6 1.1
Medium Dunes 9 > H > 3 1.2 to 1.1
Large Dunes H > 9 1.3

3.5.4.3 Scour for Complex Pier Foundations

Most pier scour research has focused on solid piers with limited attention to determining scour depths for (1) pile groups, (2) pile groups and pile caps, or (3) pile groups, pile caps and solid piers exposed to the flow. In the
general case, the flow could be obstructed by three substructure elements (scour-producing components), which
include the pier stem, the pile cap or footing, and the pile group. Reference to FHWA's HEC-18 is suggested for methods and equations to determine scour depths for complex pier foundations.

3.5.4.4 Multiple Columns Skewed to the Flow

For multiple columns (illustrated as a group of cylinders in Figure 1-3-18) skewed to the flow, the scour depth
depends on the spacing between the columns. The correction factor for angle of attack would be smaller than for a solid pier. Raudkivi in discussing effects of alignment states "...the use of cylindrical columns would produce a shallower scour; for example, with five-diameter spacing the local scour can be limited to about 1.2 times the local scour at a single cylinder."
In application of EQ 16 with multiple columns spaced less than 5 pier diameters apart, the pier width 'a' is the total projected width of all the columns in a single bent, normal to the flow angle of attack (Figure 1-3-18). For
example, three 2.0 m (6.6 ft) cylindrical columns spaced at 10.0 m (33 ft) would have an 'a' value ranging between 2.0 and 6.0 m (6.6 and 33 ft), depending upon the flow angle of attack. **This composite pier width would be used in EQ 16 to determine depth of pier scour.** The correction factor K_1 in EQ 16 for the multiple column would be 1.0 regardless of column shape. The coefficient K_2 would also be equal to 1.0 since the effect of skew would be accounted for by the projected area of the piers normal to the flow.
The scour depth for multiple columns skewed to the flow can also be determined by determining the K_2 factor using EQ 19 and using it in EQ 16. The width "a" in EQ 16 would be the width of a single column.
If the multiple columns are spaced 5 diameter or greater apart; and debris is not a problem, limit the scour depths to a maximum of 1.2 times the local scour of a single column.
The depth of scour for a multiple column bent will be analyzed in this manner except when addressing the effect of debris lodged between columns. If debris is evaluated, it would be logical to consider the multiple columns and debris as a solid elongated pier. The appropriate L/a value and flow angle of attack would then be
used to determine K_2 in EQ 19.

Figure 1-3-18. Multiple Columns Skewed to the Flow

3.5.4.5 Scour From Debris on Piers

Debris (or ice) lodged on a pier can increase local scour at a pier. The debris may increase pier width and deflect
a component of flow downward. This increases the transport of sediment out of the scour hole. When floating debris or ice is lodged on the pier, the scour depth can be estimated by assuming that the pier width is larger than the actual width. The problem is in determining the increase in pier width to use in the pier scour

equation. At large depths, the effect of the debris or ice on scour depth should diminish. Debris and ice effects on contraction scour can also be accounted for by estimating the amount of flow blockage (decrease in width of
the bridge opening) in the equations for contraction scour. Limited field measurements of scour at ice jams indicate the scour can be as much as 10 to 30 ft.

3.5.4.6 Topwidth of Scour Holes

The topwidth of a scour hole in cohesionless bed material from one side of a pier or footing can be estimated from the following equation:
where:
The angle of repose of cohesionless material in air ranges from about 30× to 44×. Therefore, if the bottom width of the scour hole is equal to the depth of scour y_s (K = 1), the topwidth in cohesionless sand would vary from 2.07 to 2.80 y_s. At the other extreme, if K = 0, the topwidth would vary from 1.07 to 1.8 y_s. Thus, the topwidth could range from 1.0 to 2.8 y_s and depends on the bottom width of the scour hole and composition of
the bed material. In general, the deeper the scour hole, the smaller the bottom width. In water, the angle of repose of cohesionless material is less than the values given for air; therefore, a topwidth of 2.0 y_s is suggested
for practical applications (Figure 1-3-19).

W = Topwidth of the scour hole from each side of the pier or footing, m (ft)
y_s = Scour depth, m (ft)
K = Bottom width of the scour hole related to the depth of scour
θ = Angle of repose of the bed material ranging from about 30° to 44°

$$W = y_s(K + \cot\theta) \quad \textbf{EQ 20}$$

3.5.5 EVALUATING LOCAL SCOUR AT ABUTMENTS (2005)

3.5.5.1 General

Scour occurs at abutments when the abutment and embankment obstruct the flow. Several causes of abutment
failures during post-flood field inspections of bridge sites have been documented:

- Overtopping of abutments or approach embankments
- Lateral channel migration or stream widening processes
- Contraction scour
- Local sour at one or both abutments

Abutment damage is often caused by a combination of these factors. Where abutments are set back from the channel banks, especially on wide floodplains, large local scour holes have been observed with scour depths of
as much as four times the approach flow depth on the floodplain. As a general rule, the abutments most vulnerable to damage are those located at or near the channel banks.
The flow obstructed by the abutment and approach railroad embankment forms a horizontal vortex starting at
the upstream end of the abutment and running along the toe of the abutment, and a vertical wake vortex at the
downstream end of the abutment (Figure 1-3-20).

Figure 1-3-19. Topwidth of Scour Hole

The vortex at the toe of the abutment is very similar to the horseshoe vortex that forms at piers, and the vortex
that forms at the downstream end is similar to the wake vortex that forms downstream of a pier. Research has
been conducted to determine the depth and location of the scour hole that develops for the horizontal (so called
horseshoe) vortex that occurs at the upstream end of the abutment, and numerous abutment scour equations have been developed to predict this scour depth.

Abutment failures and erosion of the fill also occur from the action of the downstream wake vortex. However, research and the development of methods to determine the erosion from the wake vortex has not been conducted. An example of abutment and approach erosion of a bridge due to the action of the horizontal and wake vortex is shown in Figure 1-3-21.

The types of failures described above are initiated as a result of the obstruction to the flow caused by the abutment and railroad embankment and subsequent contraction and turbulence of the flow at the abutments.

There are other conditions that develop during major floods, particularly on wide floodplains, that are more difficult to foresee but that need to be considered in the hydraulic analysis and design of the substructure:

Figure 1-3-20. Schematic Representation of Abutment Scour

Figure 1-3-21. Scour of Bridge Abutment and Approach Embankment

- Gravel pits on the floodplain upstream of a structure can capture the flow and divert the main channel flow out of its normal banks into the gravel pit. This can result in an adverse angle of attack of the flow on the downstream embankment with subsequent breaching of the embankment and/ or failure of the abutment.
- Levees can become weakened and fail with resultant adverse flow conditions at the bridge abutment.
- Debris can become lodged at piers and abutments and on the bridge superstructure, modifying flow conditions and creating adverse angles of attack of the flow on bridge piers and abutments.

3.5.5.2 Designing for Scour at Abutments

The preferred design approach is to place the abutment foundation on scour resistant rock or on deep foundations. Present technology has not developed sufficiently to provide reliable abutment scour estimates for all hydraulic flow conditions that might be reasonably expected to occur at an abutment. Therefore, engineering judgment is required in designing foundations for abutments. In many cases, foundations can be designed with shallower depths than predicted by the equations when they are protected with rock riprap and/or with a guide bank placed upstream of the abutment.

The potential for lateral channel migration, long-term degradation and contraction scour should be considered
in setting abutment foundation depths near the main channel. It is recommended that the abutment scour equations presented in this section be used to develop insight as to the scour potential at an abutment.

3.5.5.3 Abutment Conditions

Abutments can be set back from the natural stream bank, placed at the bankline or, in some cases, actually set into the channel itself. Common designs include stub abutments placed on spill-through slopes, and vertical wall abutments, with or without wingwalls. Scour at abutments can be live-bed or clear-water scour. The bridge and approach embankment can cross the stream and floodplain at a skew angle and this will have an effect on flow conditions at the abutment. Finally, there can be varying amounts of overbank flow intercepted by the approaches to the bridge and returned to the stream at the abutment. More severe abutment scour will occur when the majority of overbank flow returns to the bridge opening directly upstream of the bridge crossing. Less severe abutment scour will occur when overbank flows gradually return to the main channel upstream of the bridge crossing.

3.5.5.4 Abutment Skew

The skew angle for an abutment (embankment) is depicted in Figure 1-3-22. For an abutment angled

downstream, the scour depth is decreased, whereas the scour depth is increased for an abutment angled upstream.

3.5.5.5 Abutment Shape

There are three general shapes of abutments: (1) spill-through abutments, (2) vertical walls without wing walls, and (3) vertical-wall abutments with wing walls (Figure 1-3-23). These shapes have varying angles to the
flow. As shown in Table 1-3-13, depth of scour is approximately double for vertical-wall abutments as compared
with spill-through abutments. Similarly, scour at vertical wall abutments with wingwalls is reduced to 82 percent of the scour of vertical wall abutments without wingwalls.

Table 1-3-13. Abutment Shape Coefficients

Description	K_1
Vertical-wall abutment	1.00
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

θ

Figure 1-3-22. Orientation of Embankment Angle, θ, to the Flow

Figure 1-3-23. Abutment Shape

3.5.5.6 Live-Bed Scour at Abutments

An equation based on field data of scour at the end of spurs in the Mississippi River (obtained by the USACE) can be used for estimating abutment scour. This field situation closely resembles the laboratory experiments for abutment scour in that the discharge intercepted by the spurs was a function of the spur length. The modified equation, is applicable when the ratio of projected abutment (embankment) length (L) to the flow depth (y_1) is greater than 25. This equation can be used to estimate scour depth (y_s) at an abutment where conditions are similar to the field conditions from which the equation was derived:
where:

For cases where the abutment (embankment) length is small in comparison to flow depth ($L/y_1 \leq 25$), the following equation for local live-bed scour can be used to estimate abutment scour at a stable spill slope when the flow is subcritical:

Where the variables are defined as for EQ 21. EQ 21 and EQ 22 are recommended for both live-bed and clearwater
abutment scour conditions.

3.5.6 TOTAL SCOUR CALCULATION PROBLEM (2005)

Figure 1-3-24 shows a cross section plot of a railroad crossing a small stream. The bridge is 50 feet long with vertical abutments and a single pier in the channel. The left and right abutments are set back 10 and 15 feet from the channel banks, respectively. The rectangular pier is 1.5 feet wide by 12 feet long and is located in the center of the bridge but not in the center of the channel. The channel bed and floodplain material is a silty sand
with a median grain size, D_{50}, equal to 0.20 mm (0.00066 ft). Figure 1-3-25 shows the channel, floodplain and railroad crossing in plan and includes the hydraulic data required for contraction scour calculations in the channel and overbank areas under the bridge. Figure 1-3-26 shows the hydraulic data for computing pier and abutment scour. Assume zero and eight degree angles of attack for the pier scour computation. The flow data are from a hydraulic analysis of the crossing for the design discharge of 1000 cfs. The railroad grade is at an

elevation of 13.0 ft, the bridge low chord is at an elevation of 10.0 ft, the water surface at the crossing is at

y_s = Scour depth, m (ft)
y_1 = Depth of flow at the abutment on the overbank or in the main channel, m (ft)
Fr = Froude Number based on the velocity and depth adjacent to and upstream of the abutment
L = Length of embankment projected normal to the flow m (ft)
K_1 = Abutment shape coefficient (from Table 1-3-13)
K_2 = Coefficient for angle of embankment to flow
$K_2 = (\theta/90)^{0.13}$ (see Figure 1-3-22 for definition of θ)

$\theta < 90°$ if embankment points downstream

$\theta > 90°$ if embankment points upstream

y
y
s Fr K K
1
0 33 1
2 4
0 55
= .
. **EQ 21**
2
0.33 1
r
0.4
1 1
s K
0.55
F K
y
1.1 L
y
y
| |)
\
| | \
(
= **EQ 22**

elevation 8.5 ft, the floodplain is at elevation of 5.0 ft and the channel invert is at an elevation of -0.9 ft. The crossing causes 0.5 feet of backwater at the cross section upstream of the bridge (water surface equals 9.0 feet).

-5
0
5
10
15
20 60 100 140 180 220 260
Station (ft)
Elevation (ft)
Water Surface
Ground and Bridge

Figure 1-3-24. Cross Section for Total Scour Problem

3.5.6.1 Main Channel Contraction Scour

Determine if the upstream channel flow is live-bed or clear-water by comparing the average channel velocity to
the critical velocity for 0.20 mm sand.

Average channel velocity

$V = Q_1/A = 700 / (25.0 \times 9.0) = 3.1$ ft/s

Critical velocity (EQ 11)

$V_c = K_u y^{1/6} D^{1/3} = 11.17\ (9.0)^{1/6}\ (0.00066)^{1/3} = 1.4$ ft/s

Approach
Channel Flow
Q_1= 700 cfs
W_1= 25 ft
y_1= 9 ft
S_1= 0.00035
Channel flow
in bridge
Q_2= 860 cfs
W_{tot}= 25 ft
W_{piers}= 1.5 ft
W_2= 23.5 ft
y_0= 8.5 ft
Left Floodplain Right Floodplain
Channel
Approach left
floodplain flow
Q = 120 cfs
Approach right
floodplain flow
Q = 180 cfs
Left overbank
flow in bridge
Q = 40 cfs
W_{tot}= 10.0 ft
W_{piers}= 0.0 ft
W= 10.0 ft
y_0= 3.5 ft
Right overbank
flow in bridge
Q = 100 cfs
W_{tot}= 15.0 ft
W_{piers}= 0.0 ft
W= 15.0 ft
y_0= 3.5 ft
Embankment Embankment

Figure 1-3-25. Hydraulic Data for Contraction Scour Calculations

The channel flow velocity is greater than the critical velocity for the bed material and the channel contraction scour is live-bed. Determine k_1 for the live-bed contraction scour equation. Compute the ratio of shear velocity
(V_*) to particle fall velocity (ω from Figure 1-3-15) to determine the mode of bed material transport and k_1.

$V_* = (g y_1 S_1)^{1/2} = (32.2 \times 9.0 \times 0.00035)^{1/2} = 0.32$ ft/s

ω = 0.025 m/s = 0.082 ft/s

V_*/ω = 0.32 / 0.082 = 3.9

The ratio of shear velocity to fall velocity is greater than 2.0 and the mode of bed material transport is mostly suspended. Therefore k_1 = 0.69.

Live-bed contraction scour (EQ 12 and EQ 13)

$y_s = y_2 - y_0$ = 11.2 - 8.5 = 2.7 ft

3.5.6.2 Right overbank contraction scour

Assume that the overbank contraction scour is clear water. This assumption can be checked by comparing the upstream floodplain flow velocity (V = 0.38 ft/s) to the critical velocity (V_c = 1.2 ft/s).

Clear-water contraction scour (EQ 14 and EQ 15)

$D_m = 1.25D_{50}$ = 1.25 x 0.00066 = 0.00083 ft

$y_s = y_2 - y_0$ = 4.8 - 3.5 = 1.3 ft

3.5.6.3 Left overbank contraction scour

$y_s = y_2 - y_0$ = 3.1 - 3.5 = -0.4 ft (actually 0.0 ft)

Negative clear-water scour indicates that there is insufficient velocity to cause erosion in the overbank area under the bridge. Therefore, the there is no contraction scour on the left overbank. Sediment will not deposit in the left overbank area under the bridge because there is no sediment in transport from the floodplain upstream (clear-water upstream floodplain flow).

11.2 ft
23.5
25.0
700
9.0 860
W
W
Q
y y Q
K 6 / 7 0.69
2
1
6 / 7
1
2
2 1
1
= |)
\
| \
(
|)
\
| \
= (|
|)
\
| | \
(
| |)

( ) ( ) 4.8 ft

0.00083 15

0.0077 100

D W

y K Q

3 / 7

2 / 3 2

3 / 7 2

2 3 / 2m

2

u

2 =

= ( )

( ) ( ) 3.1 ft

0.00083 10

0.0077 40

D W

y K Q

3 / 7

2 / 3 2

3 / 7 2

2 3 / 2m

2

u

2 =

= | ⌋
⌉
| ⌊
⌈
=

3.5.6.4 Pier Scour

Pear Scour for zero degree angle of attack (EQ 16)

Although the pier is not in the center of the channel, use the maximum channel velocity since the channel has the potential to migrate laterally. The pier shape is square end (K_1 = 1.1), the angle of attack is 0 degrees (K_2 = 1.0), and the bed condition is small dunes or plane bed (K_3 = 1.1).

Pier scour for 8 degree angle of attack (EQ 16 and EQ 19)

The pier shape is not included because angle of attack is greater than 5 degrees (K_1 = 1.0), the angle of attack is 8 degrees (K_2 = 1.5 from Table 1-3-11 for 8 degrees and L/a = 12/1.5 = 8 or use EQ 19), and the bed condition
is small dunes or plane bed (K_3 = 1.1).

$K_2 = (\cos\theta + (L / a)\sin\theta)^{0.65} = (\cos 8 + (12 / 1.5)\sin 8)^{0.65} = 1.6$

Note: The difference between Table 1-3-11 and EQ 19 is due to linear interpolation and rounding when using the table.

Left Floodplain Channel Right Floodplain
Flow at
left abutment
y_1 = 3.5 ft
V = 1.1 ft/s
Flow at
right abutment
y_1 = 3.5 ft
V = 1.9 ft/s
70 ft
80 ft
105 ft
120 ft
Embankment Embankment
Flow at Pier
y_1(max) = 9.4 ft
V_1(max) = 4.3 ft/s

Figure 1-3-26. Hydraulic Data for Local Scour Calculations

( ) ( ) 0.25

32.2 x 9.4
4.3
gy
Fr V 1/ 2 1/ 2
1
1
1 = = =

( ) (0.25) 3.8 ft

9.4
Fr 9.4 x 2.0 x1.1x1.0 x1.1 1.5
y
y y 2.0K K K a 0.43

0.65
0.43
1
0.65
1

s 1 1 2 3 =

=

=

Roadway and Ballast

3.5.6.5 Right Abutment Scour

Check the ratio of embankment length (L) to flow depth at the abutment (y_1) to determine which equation is applicable. $L / y_1 = 105 / 3.5 = 30 > 25$. Use EQ 21. Abutment shape is vertical-wall ($K_1 = 1.0$) and the embankment is perpendicular to flow ($\theta = 90$, $K_2 = 1.0$).

For deep abutment scour, protect the abutment with riprap (Article 3.6.4.2). Also consider using a spill through slope to reduce scour to 8.0 ft and provide riprap protection.

3.5.6.6 Left Abutment Scour

Check the ratio of embankment length (L) to flow depth at the abutment (y_1) to determine which equation is applicable. $L / y_1 = 70 / 3.5 = 20 < 25$. Use EQ 22. Abutment shape is vertical-wall ($K_1 = 1.0$) and the embankment is perpendicular to flow ($\theta = 90$, $K_2 = 1.0$).

For deep abutment scour, protect the abutment with riprap (Article 3.6.4.2). Also consider using a spill through slope to reduce scour to 6.0 ft and provide riprap protection.

3.5.6.7 Plot Total Scour

The total scour is the sum of long-term degradation, contraction and local scour. Figure 1-3-27 shows the contraction and local scour plotted in the bridge cross section. If long-term degradation is expected, then it should be included in the main channel prior contraction and local scour. If the channel could migrate laterally, then the maximum scour can occur at piers located in the overbank areas. The local scour hole side slopes are shown at a 1.25H:1V slope due to the cohesive channel bed and floodplain materials. The abutment scour could be eliminated with well designed riprap and the abutment scour could be substantially reduced if the abutment included a spill-through slope. However, the spill-through slope would reduce also bridge open area. The channel pier should be designed for the maximum potential scour.

( ) (0.25) 5.5 ft

9.4

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y

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0.43

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3.5.7 REFERENCES FOR SECTION 3.5 (2005)

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-15
-10
-5
0
5
10
15
100 110 120 130 140 150 160 170
Station (ft)
Elevation (ft)
Water Surface
Ground and Bridge
Contraction Scour
Local Scour

Figure 1-3-27. Total Scour Plot

U.S. Army Corps of Engineers, 1993. "Scour and Deposition in Rivers and Reservoirs," User's Manual, HEC-6, Hydrologic Engineering Center, Davis, CA.

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SECTION 3.6 PROTECTING ROADWAY AND BRIDGES FROM SCOUR

Adequate protection against floods and washouts is essential not only for maintenance of dependable service, but also to avoid heavy expenditures to replace damaged facilities and restore operation.

3.6.1 EMBANKMENT (2005)

3.6.1.1 General – Risks and Possible Damage

Water overflowing the embankment, either from a direct flow or backwater, frequently results in damage to the
railroad. This damage may be as severe as a washout or less apparent in other forms, such as, a loss of the shoulder, a steepening of the embankment, a loss of crib or shoulder ballast, or a softening of the subgrade's support characteristics. Damage resulting from sloughing and slides is usually more severe as the water recedes from a saturated embankment. Loose, fine-grained, cohesionless soils are more susceptible to sloughing. In general, soil conditions, vegetation, and the rapidity at which the water recedes are primary factors in determining the risk of sloughing.

3.6.1.2 Temporary Protection Measures

Temporary protection of the railway embankment section is sometimes necessary, particularly in flood events
where immediate action is necessary and time constraints do not permit implementation of a permanent solution. Periodic and close track inspections of flood and washout susceptible areas and identification of high
risk locations will be a beneficial first step in determining the appropriate remedial repair.
Temporary protection of potential overflow slopes and fill sections subject to erosion and sloughing can be provided by placement of an armor of heavy weight material, not easily displaced by floodwaters, such as largesized
stone (riprap) or sandbags. In blanketing the slopes, it is critical that the toe be adequately protected to minimize the risk of base scour and possible embankment failure. Raising the embankment shoulder with riprap and sandbags can also be a suitable means for temporary relief.

3.6.1.3 Permanent Protection Measures

In overflow territories, care must be taken to review the adequacy of design, location and construction of existing drainageways and make appropriate corrections if deficiencies are found. Sufficient waterway capacity
is essential to minimize heading during floods and, if necessary, provisions should be made for additional relief
openings to handle the flow. The impact of runoff from neighboring facilities, existing and proposed, must also
be assessed. Input from applicable local, state, or federal authorities should be sought in these preliminary drainage assessments.
Selection of the optimal permanent protection measure should be done on a site-specific basis and will depend
on many factors, including service requirements, severity and extent of the damage potential, embankment soil
characteristics, and economic considerations. A subsurface exploration of the area in question, performed during the preliminary stages, can many times generate valuable information and aid in the selection and design process.

Natural Waterways

In general, depending upon service requirements, a track raise is the best assurance for reliable operation. Embankments subject to severe side erosion can be protected by relocation of the track and/or channel, or construction of revetments as discussed in Article 3.6.4.5. In overflow bottoms where either a channel change,
installation of additional openings, or a track raise or relocation do not afford sufficient relief, consideration

should be given to facing the downstream side of the embankments at least at critical locations with riprap or other suitable means of protection. Covering erosion-susceptible slopes with a thick vegetative cover can furthermore provide protection by impeding surface erosion.

On light traffic density lines where the aforementioned extensive measures cannot be economically justified, consideration might be given to anchoring the track to the roadbed, at designated locations throughout the overflow area, utilizing cable tied to rail, timber pile, screw anchors driven in the roadbed or hot asphalt impregnated ballast. Under these conditions use of a heavy course ballast tends to reduce the incidence of ballast displacement. When using this last method of protection, the railroad is accepting the risk of traffic disruption due to flooding and washouts.

3.6.2 BRIDGES (2005)

3.6.2.1 General – Risks and Possible Damage

Protection against flood damage for structures calls for resourcefulness during the immediate flooding threat, as well as during the implementation of permanent protection measures. Temporary measures should be given consideration to prevent both minor and major damage. Minor damage can be categorized as scour on the shoulders or behind the abutments, debris hung up in the waterway opening, overtopping, and various other damage that can be immediately detected and repaired. Major damage includes items such as contamination of ballast decks and track beds, scouring around piling, piers, foundations, and backwalls; channel changes resulting in silting or bypassing the structure; culvert piping or joint separation; etc.

3.6.2.2 Temporary Protection Measures

The need for temporary protection should be considered not only prior to and during floods, but also when the structure is under construction. Temporary measures to consider during or immediately preceding a flood include, identification of high-risk areas, frequent inspection, remove or pass debris through the structure to avoid accumulation, and the placement of riprap or sandbags. The following are temporary measures to be considered when the structure is in the design or construction stage; all the measures considered previously, and others such as fence jetties, rock jetties, and channel cutoffs.

3.6.2.3 Permanent Protection Measures

Permanent protection measures require that sound engineering principles be employed to protect the structure from flood damage and allow its continued function as designed. Bridges and culverts must be designed with sufficient waterway opening to handle the design storm. In addition, both structures must be designed with an adequate opening to pass the anticipated debris. When conditions change in the upstream basin, some of the measures detailed in various articles in Article 3.6.4 may need to be incorporated in the structure protection plan. Permanent protection might also include underwater or other inspections of potential problem areas.

3.6.3 COUNTERMEASURE SELECTION (2005)

3.6.3.1 General

A countermeasure is defined as a measure incorporated into a stream crossing system to monitor, control, inhibit, change, delay, or minimize stream and bridge stability problems (HEC-23). Countermeasures may be installed at the time of railroad construction or retrofitted to resolve stability problems at existing crossings.

Retrofitting is good economics and good engineering practice in many locations because the magnitude, location, and nature of potential stability problems are not always discernible at the design stage, and indeed, may take a period of several years to develop.

A countermeasure does not need to be a separate structure, but may be an integral part of the roadbed. For example, relief bridges on floodplains are countermeasures which alleviate scour from flow contraction at the bridge over the stream channel. Some features that are integral to the railroad design serve as countermeasures to minimize stream stability problems. Abutments and piers oriented with the flow reduce local scour and contraction scour. Also, reducing the number of piers and/or setting back the abutments reduces contraction scour.

Countermeasures which are not integral to the embankment may serve one function at one location and a

different function at another. For example, bank revetment may be installed to control bank erosion from meander migration, or it may be used to stabilize streambanks in the contracted area at a bridge. Other countermeasures are useful for one function only. This category of countermeasures includes spurs constructed in the stream channel to control meander migration.

A countermeasure matrix (Table 1-3-14) has been developed which lists most countermeasures presently in use for stream instability and scour problems and summarizes river environmental factors that influence the selection of a countermeasure for a specific problem (HEC-23). In selecting a countermeasure it is necessary to evaluate how the stream might respond to the countermeasure, and also how the stream may respond as the result of the activities of other parties.

3.6.3.2 Overview of the Countermeasure Matrix

A wide variety of countermeasures have been used to control channel instability and scour at bridge foundations. The countermeasure matrix, presented in Table 1-3-14, is organized to highlight the various groups of countermeasures and to identify their individual characteristics. The left column of the matrix lists types of countermeasures in groups. In each row of the matrix, distinctive characteristics of a particular countermeasure are identified. The matrix identifies most countermeasures in use today and lists information on their functional applicability to a particular problem, their suitability to specific river environments, the general level of maintenance resources required, and which states have experience with specific countermeasures. Finally, a reference source for design guidelines is noted, where available.

Countermeasures have been organized into groups based on their functionality with respect to scour and stream instability. The three main groups of countermeasures are: **hydraulic countermeasures**, **structural countermeasures** and **monitoring**. The following outline identifies the countermeasure groups in the matrix:

Group 1. Hydraulic Countermeasures

• Group 1.A: River training structures

– Transverse structures

– Longitudinal structures

– Areal structures

• Group 1.B: Armoring countermeasures

– Revetment and Bed Armor

• Rigid

• Flexible/articulating

– Local armoring

Group 2. Structural Countermeasures

• Foundation strengthening

• Pier geometry modification

Group 3. Monitoring

• Fixed Instrumentation

• Portable instrumentation

• Visual Monitoring

3.6.3.3 Countermeasure Groups

Group 1. Hydraulic Countermeasures

Hydraulic countermeasures are those which are primarily designed either to modify the flow or resist erosive forces caused by the flow. Hydraulic countermeasures are organized into two groups: **river training structures and armoring countermeasures**. The performance of hydraulic countermeasures is dependent on design considerations such as filter requirements and edge treatment.

Group 1.A River Training Structures. River training structures are those which modify the flow. River training structures are distinctive in that they alter hydraulics to mitigate undesirable erosional and/or depositional conditions at a particular location or in a river reach. River training structures can be constructed
of various material types and are not distinguished by their construction material, but rather, by their orientation to flow. River training structures are described as **transverse**, **longitudinal** or **areal** depending on their orientation to the stream flow.

• **Transverse river training structures** are countermeasures which project into the flow field at an angle or perpendicular to the direction of flow.

• **Longitudinal river training structures** are countermeasures which are oriented parallel to the flow field or along a bankline.

• **Areal river training structures** are countermeasures which cannot be described as transverse or longitudinal when acting as a system. This group also includes countermeasure "treatments" which have areal characteristics such as channelization, flow relief, and sediment detention.

Table 1-3-14. Stream Instability and Bridge Scour Countermeasures Matrix

Table 1-3-14. Stream Instability and Bridge Scour Countermeasures Matrix (Continued)

Group 1.B Armoring Countermeasures. Armoring countermeasures are distinctive because they resist the erosive forces caused by a hydraulic condition. Armoring countermeasures do not necessarily alter the hydraulics of a reach, but act as a resistant layer to hydraulic shear stresses providing protection to the more erodible materials underneath. Armoring countermeasures generally do not vary by function, but vary more in
material type. Armoring countermeasures are classified by two functional groups: **revetments and bed armoring** or **local scour armoring.**

• **Revetments and bed armoring** are used to protect the channel bank and/or bed from erosive/hydraulic forces. They are usually applied in a blanket type fashion for areal coverage. Revetments and bed armoring can be classified as either **rigid** or **flexible/articulating**. **Rigid** revetments and bed armoring are typically impermeable and do not have the ability to conform to changes in the supporting surface. These countermeasures often fail due to undermining. **Flexible/ articulating** revetments and bed armoring can conform to changes in the supporting surface and adjust to settlement. These countermeasures often fail by removal and displacement of the armor material.

• **Local scour armoring** is used specifically to protect individual substructure elements of a bridge from local scour. Generally, the same material used for revetments and bed armoring is used for local armoring, but these countermeasures are designed and placed to resist local vortices created by obstructions to the flow.

Group 2. Structural Countermeasures

Structural countermeasures involve modification of the bridge structure (foundation) to prevent failure from scour. Typically, the substructure is modified to increase bridge stability after scour has occurred or when a bridge is assessed as scour critical. These modifications are classified as either **foundation strengthening** or **pier geometry modifications.**

• **Foundation strengthening** includes additions to the original structure which will reinforce and/or extend the foundations of the bridge. These countermeasures are designed to prevent failure when the channel bed is lowered to an expected scour elevation, or to restore structural integrity after scour has occurred. Design and construction of bridges with continuous spans provide redundancy against catastrophic failure due to substructure displacement as a result of scour. Retrofitting a simple span bridge with continuous spans could also serve as a countermeasure after scour has occurred or when a bridge is assessed as scour critical.

• **Pier geometry modifications** are used to either reduce local scour at bridge piers or to transfer scour to another location. These modifications are used primarily to minimize local scour.

Group 3. Monitoring

Monitoring describes activities used to facilitate early identification of potential scour problems. Monitoring

could also serve as a continuous survey of the scour progress around the bridge foundations. Monitoring allows
for action to be taken before the safety of the railroad is threatened by the potential failure of a bridge. Monitoring can be accomplished with instrumentation or visual inspection. A well designed monitoring program can be a very cost-effective countermeasure. Two types of instrumentation are used to monitor bridge
scour: **fixed instruments** and **portable instruments**.

- **Fixed instrumentation** describes monitoring devices which are attached to the bridge structure to detect scour at a particular location. Typically, fixed monitors are located at piers and abutments. The number and location of piers to be instrumented should be defined, as it may be impractical to place a fixed instrument at every pier and abutment on a bridge. Instruments such as sonar monitors can be used to provide a timeline of scour, whereas instruments such as magnetic sliding collars can only be used to monitor the maximum scour depth. Data from fixed instruments can be downloaded manually at the site or it can be telemetered to another location.
- **Portable instrumentation** describes monitoring devices that can be manually carried and used along a bridge and transported from one bridge to another. Portable instruments are more cost effective in monitoring an entire bridge than fixed instruments; however, they do not offer a continuous watch over the structure. The allowable level of risk will affect the frequency of data collection using portable instruments.

- **Visual inspection** describes standard monitoring practices of inspecting the bridge on a regular interval and increasing monitoring efforts during high flow events (flood watch). Typically, bridges are inspected on an annual schedule. Where stream stability is questionable, channel bed elevations at each pier location can be recorded during the annual inspection. The channel bed elevations should be compared with historical cross sections to identify changes in bed elevations due to degradation or lateral migration. Channel elevations should also be taken during and after high flow events. If measurements cannot be safely collected during a high flow event, the engineer should determine if the bridge is at risk and if train operation restrictions are necessary. Underwater inspections of the foundations could be used to supplement the visual inspection after a flood.

3.6.3.4 Countermeasure Characteristics

The countermeasure matrix (Table 1-3-14) was developed to identify distinctive characteristics for each type of
countermeasure. Five categories of countermeasure characteristics were defined to aid in the selection and implementation of countermeasures:

- Functional Applications
- Suitable River Environment
- Maintenance
- Installation/Experience by State
- Design Guidelines Reference

These categories were used to answer the following questions:

- For what type of problem is the countermeasure applicable?
- In what type of river environment is the countermeasure best suited or, are there river environments where the countermeasure will not perform well?
- What level of resources will need to be allocated for maintenance of the countermeasure?
- What states or regions in the U.S. have experience with this countermeasure?
- Where do I obtain design guidance reference material?

Functional Applications

The functional applications category describes the type of scour or stream instability problem for which the countermeasure is prescribed. The five main categories of functional applications are local scour at abutments
and piers, contraction scour, and vertical and lateral instability. Vertical instability implies the long-term processes of aggradation or degradation over relatively long river reaches, and lateral instability involves a long-term process of channel migration and bankline erosion problems. To associate the appropriate

countermeasure type with a particular problem, filled circles, half circles and open circle are used in the matrix
as described below:

● **well suited/primary use -** the countermeasure is well suited for the application; the countermeasure
has a good record of success for the application; the countermeasure was implemented primarily for this application.

● **possible application/secondary use -** the countermeasure can be used for the application; the countermeasure has been used with limited success for the application; the countermea

sure was implemented primarily for another application but also can be designed to function for this application.

In addition, this symbol can identify an application for which the countermeasure has performed successfully and was implemented primarily for that application, but there is only a limited amount of data on its performance and therefore the application cannot be rated as well suited.

O **unsuitable/rarely used -** the countermeasure is not well suited for the application; the countermeasure has a poor record of success for the application; the countermeasure was not intended for this application.

N/A not applicable - the countermeasure is not applicable to this functional application.

Suitable River Environment

This category describes the characteristics of the river environment for which a given countermeasure is best suited or under which there would be a reasonable expectation of success. Conversely, this category could indicate conditions under which experience has shown a countermeasure may not perform well. The river environment characteristics that can have a significant effect on countermeasure selection or performance are:

- River type
- Stream size (width)
- Bend radius
- Flow velocity
- Bed material
- Ice/debris load
- Bank condition
- Floodplain (width)

For each environmental characteristic, a qualitative range is established (e.g., stream size: **W**ide, **M**oderate, or **S**mall) to serve as a suitability discriminator. While most characteristics are self explanatory, both HEC-20 ("Stream Stability at Highway Structures") and HDS 6 ("River Engineering for Highway Encroachments") provide guidance on the range and definitions of these characteristics of the river environment. In the context of this matrix, the bank condition characteristic (**V**ertical, **S**teep, or **F**lat) considers the effectiveness of a given countermeasure to **protect** a bank with that configuration, **not** the suitability for installation of the countermeasure **on** a bank with that configuration.

Where a block is **checked** for a given countermeasure under an environmental characteristic, the countermeasure is considered suitable or has been applied successfully for the full range of that environmental characteristic.

The checked block means that the characteristic **does not influence** the selection of the countermeasure, i.e., the countermeasure is suitable for the full range of that characteristic. For example, **guide banks** have been applied successfully in braided, meandering, and straight streams; however, **bendway weirs/stream barbs** are most suitable for installation on meandering streams.

Maintenance

The maintenance category identifies the estimated level of maintenance that may need to be allocated to service
the countermeasure. The ratings in this category range from **"Low"** to **"High"** and are subjective. The ratings represent the relative amount of resources required for maintenance with respect to other countermeasures within
the matrix shown in Table 1-3-14. A low rating indicates that the countermeasure is relatively maintenance free,
a moderate rating indicates that some maintenance is required, and a high rating indicates that the countermeasure requires more maintenance than most of the countermeasures in the matrix.

Installation/Experience by State Departments of Transportation

This category identifies states (or a region) where a particular countermeasure has been installed. These listings may not include all of the states which have used a particular countermeasure. Certain countermeasures are used in many states. These countermeasures have a listing of "Widely Used" in this category. Both successful, and unsuccessful experiences are reflected by the listing.

Design Guideline Reference

Reference manuals which provide guidance in countermeasure design have been developed by government agencies through research programs. The FHWA has produced a wealth of information through the federally coordinated program of highway research and development. The design guideline reference column identifies
reference manuals where guidance on design of the countermeasures can be obtained. The references are symbolized by numbers in this column. The numbers correspond to the numbers of the references listed on the
second page of the matrix. Countermeasures for which design guidelines are provided in HEC-23 are referenced using **DG#**, where # represents a number assigned to the design guideline (see also Section 3.6.4).

3.6.3.5 Summary

The countermeasures matrix is convenient reference guide on a wide range of countermeasures applicable to scour and stream stability problems. An engineering plan to install countermeasures should provide conceptual design and cost information on several alternative countermeasures, with a recommended alternative based on a variety of engineering, environmental and cost factors. The countermeasures matrix is a
good way to begin identifying and prioritizing possible alternatives. The information provided in the matrix related to functional applications, suitable river applications and maintenance issues should facilitate preliminary selection of feasible alternatives prior to more detailed investigation.

3.6.3.6 Selection of Countermeasures for Stream Instability

The selection of an appropriate countermeasure for a specific bank erosion problem is dependent on factors such as the erosion mechanism, stream characteristics, construction and maintenance requirements, potential
for vandalism, and costs. Perhaps more important, however, is the effectiveness of the measure selected in performing the required function.

Protection of an existing bank line may be accomplished with revetments, spurs, retardance structures, longitudinal dikes, or bulkheads (Table 1-3-14). Spurs, longitudinal dikes, and area retardance structures can be used to establish a new flow path and channel alignment, or to constrict flow in a channel. Because of their high cost, bulkheads may be appropriate for use only where space is at a premium. Channel relocation may be used separately or in conjunction with other countermeasures to change the flow path and flow orientation.

Erosion Mechanism

Bank erosion mechanisms are surface erosion and/or mass wasting. Surface erosion is the removal of soil particles by the velocity and turbulence of the flowing water. Mass wasting is by slides, rotational slip, piping

and block failure. In general slides, rotational slip and block failure result from the bank being undercut by the
flow. Also, seepage force of the pore water in the bank is another factor that can cause surface erosion or mass
wasting. The type of mechanism is determined by the magnitude of the erosive forces of the water, type of bed
and bank material, vegetation, and bed elevation stability of the stream.

Stream Characteristics

Stream characteristics that influence the selection of countermeasures include (see also Table 1-3-14):

- Channel width
- Bank height
- Channel configuration
- Channel material
- Vegetative cover
- Sediment transport condition
- Bend radii
- Channel velocities and flow depth
- Ice and debris
- Floodplain characteristics

Channel Width. Channel width influences the use of bendway weirs and other spur-type countermeasures. On smaller streams (<250 feet wide), flow constriction resulting from the use of spurs may cause erosion of the opposite
bank. However, spurs can be used on small channels where the purpose is to shift the location of the channel.

Bank Height. Low banks (<10 feet) may be protected by any of the countermeasures, including bulkheads. Medium height banks (from 10 to 20 feet) may be protected by revetment, retardance structures, spurs, and longitudinal dikes.

High banks (>20 feet) generally require revetments used alone or in conjunction with other measures.

Channel Configuration. Spurs and jack fields have been successfully used as a countermeasure to control the location
of the channel in meandering and braided streams. Also, bulkheads, revetments, and riprap have been used to control
bank erosion resulting from stream migration. On anabranching streams, revetments, riprap, and spurs have been used to control bank erosion and channel shifting. Also, channels that do not carry large flows can and have been closed off.

Channel Material. Spurs, revetments, riprap, jack fields, or check dams can be used in any type of channel material if they are designed correctly. However, jack fields should only be placed on streams that carry appreciable debris and sediment in order for the jacks to cause deposition and eventually be buried.

Bank Vegetation. Vegetation such as willows can enhance the performance of structural countermeasures and may, in some cases, reduce the level of structural protection needed. Meander migration and other bank erosion mechanisms are accelerated on many streams in reaches where vegetation has been cleared.

Sediment Transport. The sediment transport conditions can be described as regime, threshold, or rigid. Regime channel beds are those which are in motion under most flow conditions, generally in sand or silt-size noncohesive materials. Threshold channel beds have no bed material transport at normal flows, but become mobile at higher flows. They may be cut through cohesive or noncohesive materials, and an armor layer of coarse-grained material can develop on the channel bed. Rigid channel beds are cut through rock or boulders and rarely or never become mobile. In general, permeable structures will cause deposition of bed material in transport and are better suited for use in regime and some threshold channels than in rigid channel conditions. Impermeable structures are more

effective than permeable structures in channels with little or no bed load, but impermeable structures can also be very effective in mobile bed conditions. Revetments can be effectively used with mobile or immobile channel beds.

Bend Radii. Bend radii affect the design of countermeasures, because some countermeasures will only function properly in long or moderate radius bends. Thus, the cost per meter (foot) of bank protection provided by a specific

countermeasure may differ considerably between short-radius and longer radius bends.
Channel Velocities and Flow Depth. Channel hydraulics affect countermeasure selection because structural stability and induced scour must be considered. Some of the permeable flow retardance measures may not be structurally stable and countermeasures which utilize piles may be susceptible to scour failure in high velocity environments.
Ice and Debris. Ice and debris can damage or destroy countermeasures and should always be considered during the selection process. On the other hand, the performance of some permeable spurs and area retardance structures is enhanced by debris where debris accumulation induces additional sediment deposition.
Floodplains. In selecting countermeasures for stream stability and scour, the amount of flow on the floodplain is an important factor. For example, if there is appreciable overbank flow, then the use of guide banks to protect abutments
should be considered. Also, spurs perpendicular to the approach embankment may be required to control erosion.

Construction and Maintenance Requirements

Standard requirements regarding construction or maintenance such as the availability of materials, construction equipment requirements, site accessibility, time of construction, contractor familiarity with construction methods, and a program of regular maintenance, inspection, and repair are applicable to the selection of appropriate countermeasures. Additional considerations for countermeasures located in stream channels include:
constructing and maintaining a structure that may be partially submerged at all times, the extent of bank disturbance which may be necessary, and the desirability of preserving streambank vegetative cover to the extent practicable.

Vandalism

Vandalism is always a maintenance concern since effective countermeasures can be made ineffective by vandals. Documented vandalism includes dismantling of devices, burning, and cutting or chopping with knives, wire cutters, and axes. Countermeasure selection or material selection for construction may be affected by concerns of vandalism.
For example, rock-filled baskets (gabions) may not be appropriate in some urban environments.

Costs

Cost comparisons should be used to study alternative countermeasures with an understanding that the measures were installed under widely varying stream conditions, that the conservatism (or lack thereof) of the designer is not accounted for, that the relative effectiveness of the measures cannot be quantitatively evaluated, and that some measures included in the cost data may not have been fully tested by floods.

3.6.3.7 Countermeasures for Meander Migration

The best countermeasure against meander migration is to locate the bridge crossing on a relatively straight reach of stream between bends. At many such locations, countermeasures may not be required for several years because of the time required for the bend to move to a location where it becomes a threat to the railroad facility. However, bend migration rates on other streams may be such that countermeasures will be required after a few years or a few flood events and, therefore, should be installed during initial construction.
Stabilizing channel banks at a railroad stream crossing can cause a change in the channel cross section and an increase in stream sinuosity upstream of the stabilized banks. Figure 1-3-28a illustrates a natural channel section in a bend with the deeper section at the outside of the bend and a gentle slope toward the inside bank resulting from point bar growth. Figure 1-3-28b illustrates the scour which results from stabilizing the outside bank of the

channel and the resulting steeper slope of the point bar on the inside of the bend. This effect must be considered in the design of the countermeasure and the bridge. It should also be recognized that the thalweg location and flow direction can change as sinuosity upstream increases.
Countermeasures for meander migration include those that:

- Protect an existing bank line
- Establish a new flow line or alignment
- Control and constrict channel flow

The classes of countermeasures identified for bank stabilization and bend control are bank revetments, spurs, retardance structures, longitudinal dikes, vane dikes, bulkheads, and channel relocations. Also, a carefully planned cutoff may be an effective way to counter problems created by meander migration. These measures may be used individually or in combination to combat meander migration at a site. Some of these countermeasures are also applicable to bank erosion from causes other than bend migration.

3.6.3.8 Countermeasures for Channel Braiding and Anabranching

Channel braiding occurs in streams with an overload of sediment, causing deposition and aggradation. As aggradation occurs, the slope of the channel increases, velocities increase, and multiple, interconnected channels

develop. The overall channel system becomes wider and multiple channels are formed as bars of sediment are deposited in the main channel (see HEC-20 or HDS 6).

Braiding can also occur where banks are easily eroded and there is a large range in discharge. The channel becomes wider at high flows, and low-flow forms multiple interconnected channels. In an anabranched stream, flow is divided by islands rather than bars, and the anabranched channels are more permanent than braided channels and generally convey more flow.

Braided channels change alignment rapidly, and are very wide and shallow even at flood flow. They present problems at bridge sites because of the high cost of bridging the complete channel system, unpredictable channel locations and flow directions, difficulties with eroding channel banks, and in maintaining bridge openings unobstructed by bars and islands.

Countermeasures used on braided and anabranched streams are usually intended to confine the multiple channels to one channel. This tends to increase the sediment transport capacity in the principal channel and encourage deposition in secondary channels. These measures usually consist of dikes constructed from the margins of the

Figure 1-3-28. Comparison of Channel Bend Cross Sections

(a) for Natural Conditions, and (b) for Stabilized Bend (after Brown)

braided zone to the channel over which the bridge is constructed. Guide banks at bridge abutments (see Article 3.6.4.3) in combination with revetment on embankment fill slopes (see Article 3.6.4.5), riprap on embankment fill slopes only, and spurs (see Article 3.6.4.4) arranged in the stream channels to constrict flow to one channel have also been used successfully.

Since anabranches are permanent channels that may convey substantial flow, diversion and confinement of an anabranched stream is likely to be more difficult than for a braided stream. The designer may be faced with a choice of either building more than one bridge, building a long bridge, or diverting anabranches into a single channel.

3.6.3.9 Countermeasures for Degradation and Aggradation

Bed elevation instability problems are common on alluvial streams. Degradation in streams can cause the loss of bridge piers in stream channels and can contribute to the loss of piers and abutments located on caving banks. Aggradation causes the loss of waterway opening in bridges and, where channels become wider because of aggrading streambeds, overbank piers and abutments can be undermined. At its worst, aggradation may cause streams to abandon their original channels and establish new flow paths which could isolate the existing bridge.

Countermeasures to Control Degradation

Countermeasures used to control bed degradation include check dams and channel linings. Check-dams and structures which perform functions similar to check-dams include drop structures, cutoff walls, and drop flumes.

A check-dam is a low dam or weir constructed across a channel to prevent upstream degradation (see Article 3.6.4.7).

Channel linings of concrete and riprap have proved unsuccessful at stopping degradation. To protect the lining, a check-dam may have to be placed at the downstream end to key it to the channel bed. Such a scheme would provide no more protection than would a check dam alone, in which case the channel lining would be redundant.

Bank erosion is a common hydraulic hazard in degrading streams. As the channel bed degrades, bank slopes become steeper and bank caving failures occur. The USACE found that longitudinal stone dikes, or rock toe-dikes, provided the most effective toe protection of all bank stabilization measures studied for very dynamic and/or actively degrading channels.

Countermeasures to Control Aggradation

Currently, measures used in attempts to alleviate aggradation problems include channelization, debris basins, bridge modification, and/or continued maintenance, or combinations of these. Channelization may include dredging and clearing channels, constructing small dams to form debris basins, constructing cutoffs to increase the local slope, constructing flow control structures to reduce and control the local channel width, and constructing relief channels to improve flow capacity at the crossing. Except for debris basins and relief channels, these measures are intended to increase the sediment transport capacity of the channel, thus reducing or eliminating

problems with aggradation. Cutoffs must be designed with considerable study as they can cause erosion and degradation upstream and deposition downstream (see Article 3.6.4.8). The most common bridge modifications are increasing the bridge length by adding spans and increasing the effective flow area beneath the structure by raising the bridge deck.

A program of continuing maintenance has been successfully used to control problems at bridges on aggrading streams. In such a program, a monitoring system is set up to survey the affected crossing at regular intervals. When some pre-established deposition depth is reached, the bridge opening is dredged or cleared of the deposited material. In some cases, this requires opening a clearing after every major flood. This solution requires surveillance and dedication to the continued maintenance of an adequate waterway under the bridge. Otherwise, it is only a temporary solution. A debris basin or a deeper channel upstream of the bridge may be easier to maintain. Continuing maintenance is not recommended if analysis shows that other countermeasures are practicable.

3.6.3.10 Selection of Countermeasures for Scour at Bridges

The selection of an appropriate countermeasure for scour at a bridge requires an understanding of the erosion mechanism producing the specific scour problem. For example, contraction scour results from a sediment imbalance across most or all of the channel while local scour at a pier or abutment results from the action of vortices at an obstruction to the flow. Degradation is a component of total scour, but is considered a channel instability problem.

3.6.3.11 Countermeasures for Contraction Scour

Severe contraction of flow at railroad stream crossings has resulted in numerous bridge failures at abutments, approach fills, and piers from contraction scour. Design alternatives to decrease contraction scour include longer bridges, relief bridges on the floodplain, and superstructures at elevations above flood stages of extreme events. These design alternatives are integral features of the facility which reduce the contraction at bridges and, therefore, reduce the magnitude of contraction scour.

The elevation of bridge superstructures is recognized as important to the integrity of the bridge because of hydraulic forces that may damage the superstructure. These include buoyancy and impact forces from ice and other floating debris. Contraction scour is another consideration in setting the superstructure elevation. When the superstructure of a bridge becomes submerged or when ice or debris lodged on the superstructure causes the flow to contract, flow may be accelerated and more severe scour can occur. For this reason, where contraction scour is of concern, bridge superstructures should be located with clearance for debris, and, if practicable, above the stage of floods larger than the design flood.

Similarly, pier design, span length, and pier location can become more important contributors to contraction scour where debris can lodge on the piers and further contract flow in the waterway. In streams which carry heavy loads of debris, longer, higher spans and solid piers will help to reduce the collection of debris. Where practicable, piers should be located out of the main current in the stream, i.e., outside the thalweg at high flow. There are numerous locations where piers occupy a significant area in the stream channel and contribute to contraction scour.

The principal countermeasure used for reducing the effects of contraction is revetment on channel banks and fill slopes at bridge abutments (see Article 3.6.4.5). However, guide banks may be used to reduce the effects of contraction by moving the site of local scour caused by the turbulence of intersecting flows and contraction away from the bridge abutment (see Article 3.6.4.3).

The potential for undesired effects from stabilizing all or any portion of the channel perimeter at a contraction should be considered. Stabilization of the banks may only result in exaggerated scour in the streambed near the banks or, in a relatively narrow channel, across the entire channel. Stabilization of the streambed may also result in exaggerated lateral scour in any size stream. Stabilization of the entire stream perimeter may result in downstream scour or failure of some portion of the countermeasures used on either the streambed or banks.

3.6.3.12 Countermeasures for Local Scour

Local scour occurs in bridge openings at piers and abutments. In general, design alternatives against structural failure
from local scour consist of measures which reduce scour depth, such as pier shape and orientation, and measures which retain their structural integrity after scour reaches its maximum depth, such as placing foundations in sound rock and using deep piling. Countermeasures which can reduce the risk from scour include riprap.

Abutments

Countermeasures for local scour at abutments consist of measures which improve flow orientation at the bridge end

and move local scour away from the abutment, as well as revetments and riprap placed on spill slopes to resist erosion.

Guide banks are earth or rock embankments placed at abutments. Flow disturbances, such as eddies and cross-flow, will be eliminated where a properly designed and constructed guide bank is placed at a bridge abutment.

Guide banks also protect the railroad embankment, reduce local scour at the abutment and adjacent piers, and move local scour to the end of the guide bank (see Article 3.6.4.3).

Local scour also occurs at abutments as a result of expanding flow downstream of the bridge, especially for bridges on wide, wooded floodplains that have been cleared for construction of the railroad. Short guide banks extending downstream of the abutment to the tree line will move this scour away from the abutment, and the trees will retard velocities so that flow redistribution can occur with minimal scour.

Revetments may consist of pervious rock or rigid concrete. Rock riprap revetment provides an effective countermeasure against erosion on spill slopes (see Article 3.6.4.2). Rigid revetments have been more successful where abutments are on the floodplain rather than in stream channels because hydrostatic pressure behind the revetments is not usually a problem. Precautions against undermining of the toe and upstream terminus of all revetments are always required (see Article 3.6.4.5).

Other countermeasures have been successfully used to inhibit scour at abutments where the abutment is located at the streambank or within the stream channel. These measures include dikes to constrict the width of braided streams and retards to reduce velocities near the streambank.

Piers

Three basic methods may be used to prevent damage from local scour at piers. The first method is to place the foundation of the structure at such a depth that the structural stability will not be at risk with maximum scour. The second is to provide protection at or below the streambed to inhibit the development of a scour hole. The third measure is to prevent erosive vortices from forming or to reduce their strength and intensity.

Streamlining the pier nose decreases flow separation at the face of the pier, reducing the strength of the horseshoe vortices which form at piers. Practical application of this principle involves the use of rounded or circular shapes at the upstream and downstream faces of piers in order to reduce the flow separation. However, flow direction can and does change with time and with stage on some streams. Piers oriented with flow direction at one stage or at one point in time may be skewed with flow direction at another. Also, flow direction changes with the passage of bed forms. In general, piers should be aligned with the main channel design flow direction and skew angles greater than 5 degrees should be avoided. Where this is not possible, a single cylindrical pier or a row of cylindrical columns will produce a lesser depth of local scour.

The tendency of a row of columns to collect debris should be considered. Debris can greatly increase scour depths. Webwalls have been used between columns to add to structural strength and to reduce the tendency to collect debris. Webwalls should be constructed at the elevation of stream flood stages which carry floating debris and extended to the elevation of the streambed. When installing a webwall as a countermeasure against debris, the potential for significantly increased scour depths should be considered if the approach flow might impinge on the wall at a high angle of attack.

Riprap is commonly used to inhibit local scour at piers at existing bridges. This practice is not recommended as an adequate substitute for foundations or piling located below expected scour depths for new or replacement bridges. It is recommended as a retrofit or a measure to reduce the risk where scour threatens the integrity of a pier (see Article 3.6.4.1). The practice of heaping stones around a pier is not recommended because experience has shown that continual replacement is usually required. Success rates have been better with alluvial bed materials where the top of the riprap was placed at or below the elevation of the streambed.

3.6.3.13 References for Section 3.6.3

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3.6.4 COUNTERMEASURE DESIGN GUIDANCE (2005)

3.6.4.1 Rock Riprap at Piers and Abutments

Introduction

Present knowledge for designing riprap at bridge piers is based on research conducted under laboratory conditions with little field verification. Flow turbulence and velocities around a pier are of sufficient magnitude

that large rocks move over time. Bridges have been lost due to the removal of riprap at piers resulting from turbulence and high velocity flow. Usually this does not happen during one storm, but is the result of the cumulative effect of a sequence of high flows. **Therefore, if rock riprap is placed as scour protection around a pier, the bridge should be monitored and inspected during and after each high flow event to insure that the riprap is stable**.

Sizing Rock Riprap at Piers

As a countermeasure for scour at piers for existing bridges, riprap can reduce the risk of failure. Riprap is not recommended as a pier scour countermeasure for new bridges. Determine the D_{50} size of the riprap using the rearranged Isbash equation to solve for stone diameter (in meters (ft), for fresh water):

where:

The effect of turbulence intensity on required rock size is illustrated in Figure 1-3-29.

D_{50} = median stone diameter, m (ft)

K = coefficient for pier shape

V = velocity on pier, m/s (ft/s)

S_s = specific gravity of riprap (normally 2.65)

g = 9.81 m/s^2 (32.2 ft/s^2)

K = 1.5 for round-nose pier

K = 1.7 for rectangular pier

$$D_{50} = \frac{0.692(KV)^2}{(S_s - 1)2g} \qquad \textbf{EQ 23}$$

To determine V multiply the average channel velocity (Q/A) by a coefficient that ranges from 0.9 for a pier near the

bank in a straight uniform reach of the stream to 1.7 for a pier in the main current of flow around a sharp bend.

(1) Provide a riprap mat width which extends horizontally at least two times the pier width, measured from the pier face.

(2) Place the top of a riprap mat at the same elevation as the streambed. Placing the bottom of a riprap mat on top of the streambed is discouraged. In all cases where riprap is used for scour control, the bridge must be monitored during and inspected after high flows.

It is important to note that it is a disadvantage to bury riprap so that the top of the mat is below the streambed because inspectors have difficulty determining if some or all of the riprap has been removed. Therefore, it is recommended to place the top of a riprap mat at the same elevation as the streambed.

(a) The thickness of the riprap mat should be three stone diameters (D_{50}) or more. In general, the bottom of the riprap blanket should be placed at or below the computed contraction scour depth.

(b) In some conditions, place the riprap on a geotextile or a gravel filter. However, if a well-graded riprap is used, a filter may not be needed. In some flow conditions it may not be possible to place a filter or if the riprap is buried in the bed a filter may not be needed.

(c) The maximum size rock should be no greater than twice the D_{50} size.

Design Example for Riprap at Existing Bridge Piers

Riprap is to be sized for an existing 6 ft diameter circular pier. The velocity was determined to be 6 ft/s using the continuity equation. The pier is located between the bank and the thalweg on a gradual bend. A velocity

multiplier of 1.2 should be used to account for pier location in the channel, since the calculated value represents
a cross section average. The computed contraction scour at the pier is approximately 3.9 ft.

Step 1. Determine D_{50} and D_{max} for the riprap protection using EQ 23.

Step 2. Extent of riprap from edge of pier = 2(6) = 12 ft.

Step 3. Depth of riprap from streambed at pier = Contraction Scour = 3.9 ft.

Step 4. Use well graded riprap such that placement of filter material under water can be avoided. The gradation should be determined using the guidance for revetments (Article 3.6.4.5). This part of the design is not conducted here.

Figure 1-3-30 presents the riprap placement resulting from the design.

$$D_{50} = \frac{0.692(KV)^2}{(S_s - 1)2g}$$

$$D_{50} = \frac{0.692\,[(1.5)(1.2)(6)]^2}{(2.65 - 1)(2)(32.2)} = 0.8\text{ft}$$

$$D_{max} = 2(0.8) = 1.6\text{ ft}$$

(SI Units)
(English Units)

Figure 1-3-29. Effect of Turbulence Intensity on Rock Size Using the Isbash Approach

References for Riprap at Piers

Lagasse, P.F., Zevenbergen, L.W., Schall, J.D., and Clopper, P.E., 2001. "Bridge Scour and Stream Instability Countermeasures – Experience, Selection, and Design Guidelines," Second Edition, Report FHWA NHI 01-003, Federal Highway Administration, Hydraulic Engineering Circular No. 23, U.S. Department of Transportation, Washington, D.C.

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3.6.4.2 Rock Riprap at Abutments

Introduction

The FHWA conducted two research studies in a hydraulic flume to determine equations for sizing rock riprap for protecting abutments from scour (Pagan 1991, Atayee 1993). The first study investigated vertical wall and spill-through abutments which encroached 28 and 56 percent on the floodplain, respectively. The second study
investigated spill-through abutments which encroached on a floodplain with an adjacent main channel (Figure 1-3-31). Encroachment varied from the largest encroachment used in the first study to a full encroachment to the edge of main channel bank. For spill-through abutments in both studies, the rock riprap consistently failed at the toe downstream of the abutment centerline (Figure 1-3-32). For vertical wall abutments, the first study consistently indicated failure of the rock riprap at the toe upstream of the centerline
of the abutment.

Field observations and laboratory studies indicate that with large overbank flow or large drawdown through a
bridge opening that scour holes develop on the side slopes of spill-through abutments and the scour can be at

the upstream corner of the abutment. In addition, flow separation can occur at the downstream side of a bridge
(either with vertical wall or spill-through abutments). This flow separation causes vertical vortices which erode
the approach embankment and the downstream corner of the abutment.

Figure 1-3-30. Placement of Pier Riprap

Sizing Rock Riprap at Abutments

For Froude Numbers (V/(gy)1/2) 0.80, the recommended design equation for sizing rock riprap for spillthrough
and vertical wall abutments is in the form of the Isbash relationship:

where:

For Froude Numbers >0.80, EQ 25 is recommended:

where:

D_{50} = median stone diameter, m (ft)

V = characteristics average velocity in the contracted section (explained below), m/s (ft/s)

S_s = specific gravity of rock riprap

g = gravitational acceleration, 9.81 m/s2 (32.2 ft/s2)

y = depth of flow in the contracted bridge opening, m (ft)

K = 0.89 for a spill-through abutment

1.02 for a vertical wall abutment

K =

=

0.61 for spill through abutments

0.69 for vertical wall abutments

EQ 24

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)}\left[\frac{V^2}{gy}\right]$$

EQ 25

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)}\left[\frac{V^2}{gy}\right]^{0.14}$$

Figure 1-3-31. Section View of a Typical Setup of Spill-through Abutment on a Floodplain With Adjacent Main Channel

In both equations, the coefficient K, is a velocity multiplier to account for the apparent local acceleration of flow
at the point of rock riprap failure. Both of these equations are envelope relationships that were forced to over predict 90 percent of the laboratory data.
A recommended procedure for selecting the characteristic average velocity is as follows:
(1) Determine the set-back ratio (SBR) of each abutment. SBR is the ratio of the set-back length to channel flow depth. The set-back length is the distance from the near edge of the main channel to the toe of abutment.
SBR = Set-back length/average channel flow depth
(a) If SBR is less than 5 for both abutments (Figure 1-3-33), compute a characteristic average velocity, Q/A, based on the entire contracted area through the bridge opening. This includes the total upstream flow, exclusive of that which overtops the railroad.
(b) If SBR is greater than 5 for an abutment (Figure 1-3-34), compute a characteristic average velocity, Q/A, for the respective overbank flow only. Assume that the entire respective overbank flow stays in the overbank section through the bridge opening.
(c) If SBR for an abutment is less than 5 and SBR for the other abutment at the same site is more than 5 (Figure 1-3-35), a characteristic average velocity determined from Step 1a for the abutment with SBR less than 5 may be unrealistically low. This would, of course, depend upon the opposite overbank discharge as well as how far the other abutment is set back. For this case, the characteristic average velocity for the abutment with SBR less than 5 should be based on the

Figure 1-3-32. Plan View of the Location of Initial Failure Zone of Rock Riprap for Spill-through Abutment

flow area limited by the boundary of that abutment and an imaginary wall located on the opposite channel bank. The appropriate discharge is bounded by this imaginary wall and the outer edge of the floodplain associated with that abutment.
(2) Compute rock riprap size from EQ 24 or EQ 25, based on the Froude Number limitation for these equations.
(3) Determine extent of rock riprap.
(a) The apron at the toe of the abutment should extend along the entire length of the abutment toe, around the curved portions of the abutment to the point of tangency with the plane of the embankment slopes.
(b) The apron should extend from the toe of the abutment into the bridge waterway a distance equal to twice the flow depth in the overbank area near the embankment, but need not exceed 25 ft (Figure 1-3-36).
(c) Spill-through abutment slopes should be protected with the rock riprap size computed from EQ 24 or EQ 25 to an elevation 2 ft above expected high water elevation for the design flood.
Upstream and downstream coverage should agree with step 3a except that the downstream riprap should extend back from the abutment 2 flow depths or 25 ft which ever is larger to protect the approach embankment. In the southeast a guide bank 50 ft long at the downstream end of the abutment to protect the downstream side of the abutment is often used.
(d) The rock riprap thickness should not be less than the larger of either 1.5 times D_{50} or D_{100}. The rock riprap thickness should be increased by 50 percent when it is placed under water to provide for the uncertainties associated with this type of placement.
(e) The rock riprap gradation and potential need for underlying filter material must be considered (see Article 3.6.4.5).

Design Example for Riprap at Bridge Abutments

Riprap is to be sized for an abutment located on the floodplain at an existing bridge. The bridge is 650 ft long, has spill through abutments on a 1V:2H side slope and 7 equally spaced spans. The left abutment is set back

from the main channel 225 ft. Given the following table of hydraulic characteristics for the left abutment size the riprap.

Step 1. Determine characteristic average velocity, V. Abutment is set back more than 5 average flow depths, therefore overbank discharge and areas are used to determine V.

V = Q/A = 7720/613.5 = 12.6 ft/s

Hydraulic Property	Value	Remarks
y (ft)	2.7	Flow depth adjacent to abutment
Q (cfs)	7,720	Discharge in left overbank
A (ft2)	613.5	Flow area of left overbank

Figure 1-3-33. Characteristic Average Velocity for SBR<5

Figure 1-3-34. Characteristic Average Velocity for SBR>5

Figure 1-3-35. Characteristic Average Velocity for SBR>5 and SBR<5

Figure 1-3-36. Plan View of the Extension of Rock Riprap Apron

Step 2. Determine the Froude Number of the flow.

$Fr = V/(gy)^{1/2} = 12.6/(32.2(2.7)) = 1.35$

Step 3. Determine the D_{50} of the riprap for the left abutment. The Froude Number is greater than 0.8, therefore, use Equation 8.3.

Step 4. Determine riprap extent and layout.

- Extent into floodplain from toe of slope = 2(2.7) = 5.4 ft
- Vertical extent up abutment slope from floodplain = 2.0 ft + 2.7 ft = 4.7 ft
- The downstream face of the embankment should be protected a distance of 25 ft from the point of tangency between the curved portion of the abutment and the plane of the embankment slope.
- Riprap mattress thickness = 1.5 (1.1) = 1.7 ft. Also, the thickness should not be less than D_{100}.
- Riprap gradation and filter requirements should be designed using Article 3.6.4.5. This portion of the design is not conducted for this example.

References for Riprap at Abutments

Atayee, A. Tamin, 1993, "Study of Riprap as Scour Protection for Spill-through Abutment," presented at the 72nd Annual TRB meeting in Washington, D.C., January.

Lagasse, P.F., Zevenbergen, L.W., Schall, J.D., and Clopper, P.E., 2001. "Bridge Scour and Stream Instability Countermeasures – Experience, Selection, and Design Guidelines," Second Edition, Report FHWA NHI 01-003, Federal Highway Administration, Hydraulic Engineering Circular No. 23, U.S. Department of Transportation, Washington, D.C.

Pagan-Ortiz, Jorge E., 1991, "Stability of Rock Riprap for Protection at the Toe of Abutments Located at the Floodplain," FHWA Research Report No. FHWA-RD-91-057, U.S. Department of Transportation, Washington, D.C.

3.6.4.3 Guide Banks

Background

When embankments encroach on wide flood plains, the flows from these areas must flow parallel to the approach embankment to the bridge opening. These flows can erode the approach embankment. A severe flow contraction at the abutment can reduce the effective bridge opening, which could possibly increase the severity of abutment and pier scour.

D_{50}
y
--------- K
$(S_s – 1)$
-------------------- V^2
gy

0.14
=
D_{50}
2.7
--------- 0.61
2.65 – 1
-------------------- 12.6^2
(32.2)(2.7)

0.14
= = 0.40

D 0.4(0.83) 0.33 50 = =

Guide banks (formerly known as spur dikes) can be used in these cases to prevent erosion of the approach embankments by cutting off the flow adjacent to the embankment, guiding streamflow through a bridge opening, and transferring scour away from abutments to prevent damage caused by abutment scour. The two major enhancements guide banks bring to bridge design are (1) reduce the separation of flow at the upstream abutment face and thereby maximize the use of the total bridge waterway area, and (2) reduce the abutment scour due to lessening turbulence at the abutment face. Guide banks can be used on both sand- and gravel-bed streams.

Principal factors to be considered when designing guide banks, are their orientation to the bridge opening, plan shape, upstream and downstream length, cross-sectional shape, and crest elevation. Bradley is used as the principal design reference for this section (Bradley 1978).

Figure 1-3-37 presents a typical guide bank plan view. It is apparent from the figure that without this guide bank overbank flows would return to the channel at the bridge opening, which can increase the severity of contraction and scour at the abutment. Note, that with installation of guide banks the scour holes which normally would occur at the abutments of the bridge are moved upstream away from the abutments. Guide banks may be designed at each abutment, as shown, or singly, depending on the amount of overbank or flood plain flow directed to the bridge by each approach embankment.

The goal in the design of guide banks is to provide a smooth transition and contraction of the streamflow through the bridge opening. Ideally, the flow lines through the bridge opening should be straight and parallel.

As in the case with other countermeasures, the designer should consider the principles of river hydraulics and morphology, and exercise sound engineering judgment.

Design Guidelines

Orientation: Guide banks should start at and be set parallel to the abutment and extend upstream from the bridge opening. If there are guide banks at each abutment, the distance between them at the bridge opening

Figure 1-3-37. Typical Guide Bank (Modified from Bradley)

should be equal to the distance between bridge abutments. Best results are obtained by using guide banks with a plan form shape in the form of a quarter of an ellipse, with the ratio of the major axis (length L_s) to the minor axis (offset) of IV:2.5H. This allows for a gradual constriction of the flow. Thus, if the length of the guide bank measured perpendicularly from the approach embankment to the upstream nose of the guide bank is denoted as L_s, the amount of expansion of each guide bank (offset), measured from the abutment parallel to the approach embankment, should be 0.4 L_s.

The plan view orientation can be determined using EQ 26, which is the equation of an ellipse with origin at the base of the guide bank. For this equation, X is the distance measured perpendicularly from the bridge approach and Y is the offset measured parallel to the approach embankment, as shown on Figure 1-3-37.

It is important that the face of the guide bank match the abutment so that the flow is not disturbed where the guide bank meets the abutment. For new bridge construction, abutments can be sloped to the channel bed at the same angle as the guide bank. For retrofitting existing bridges modification of the abutments or wing walls may be necessary.

Length: For design of guide banks, the length of the guide bank, L_s must first be determined. This can be easily determined using a nomograph which was developed from laboratory tests performed at Colorado State University and from field data compiled by the USGS. For design purposes the use of the nomograph involves the following parameters:

A nomograph is presented in Figure 1-3-38 to determine the projected length of guide banks. This nomograph should be used to determine the guide bank length for designs greater than 50 ft and less than 250 ft. If the nomograph indicates the length required to be greater than 250 ft the design should be set at 250 ft. It is recommended that the minimum length of guide banks be 50 ft. An example of how to use this nomograph is presented in the next section.

Q = total discharge of the stream, m^3/s (ft^3/s)

Q_f = lateral or flood plain discharge of either flood plain intercepted by the embankment, m^3/s (cfs) (ft^3/s)

Q_A = discharge in 30 m (100 ft) of stream adjacent to the abutment, m^3/s (ft^3/s)

b = length of the bridge opening, m (ft)

A_{n2} = cross-sectional flow area at the bridge opening at normal stage, m^2 (ft^2)

V_{n2} = Q/A_{n2} = average velocity through the bridge opening, m/s (ft/s)

Q_f/Q_A = guide bank discharge ratio

L_s = projected length of guide bank, m (ft)

X
L
Y
L s s
2
2
2
0 4 2

+ = 1

(.) **EQ 26**

FHWA practice has shown that many guide banks have performed well using a standardized length of 150 ft. Based on this experience, guide banks of 150 ft in length should perform very well in most locations. Even shorter guide banks have been successful if the guide bank intersects the tree line. If the main channel is equal to or less than 100 ft use the total main channel flow in determining the guide bank discharge ratio (Q_f/Q_A).

Crest Height: As with deflection spurs, guide banks should be designed so that they will not be overtopped at the design discharge. If this were allowed to occur, unpredictable cross flows and eddies might be generated, which could scour and undermine abutments and piers. In general, a minimum of 2 ft of freeboard, above the design water surface elevation should be maintained.

Shape and Size: The cross-sectional shape and size of guide banks should be similar to deflector, or deflector/retarder spurs discussed in Article 3.6.4.4. Generally, the top width is 10 to 13 ft, but the minimum

Figure 1-3-38. English Version of Nomograph to Determine Guidebank Length (after Bradley)

width is 3 ft when construction is by drag line. The upstream end of the guide bank should be round nosed. Side slopes should be 1V:2H or less.

Downstream Extent: In some locations, guide banks have been extended downstream of the abutments to minimize scour due to rapid expansion of the flow at the downstream end of the abutments. These downstream guide banks are sometimes called "heels." If the expansion of the flow is too abrupt, a shorter guide bank, which usually is less than 50 ft long, can be used downstream. Downstream guide banks should also start at and start parallel to the abutment and the distance between them should enlarge as the distance from the abutment of the bridge increases.

In general, downstream guide banks are a shorter version of the upstream guide banks. Riprap protection, crest height and width should be designed in the same manner as for upstream guide banks.

Riprap: Guide banks are constructed by forming an embankment of soil or sand extending upstream from the abutment of the bridge. To inhibit erosion of the embankment materials, guide banks must be adequately protected with riprap or stone facing.

Rock riprap should be placed on the stream side face as well as around the end of the guide bank. It is not necessary to riprap the side of the guide bank adjacent to the railroad approach embankment. As in the case of spurs, a gravel, sand, or geotextile filter may be required to protect the underlying embankment material (see HEC-11 and Article 3.6.4.5). Riprap should be extended below the bed elevation to a depth as recommended in Article 3.6.4.5 (below the combined long-term degradation and contraction scour depth), and extend up the face of the guide bank to 2 ft above the design flow. Additional riprap should be placed around the upstream end of the guide bank so to protect the embankment from scour.

As in the case of spurs, it is important to adequately tie guide banks into the approach embankment for guide banks on non-symmetrical railroad crossings. Hydraulics of Bridge Waterways (Bradley 1978) states:

"From meager testing done to date, there is not sufficient evidence to warrant using longer dikes (guide banks) at either abutment on skewed bridges. Lengths obtained from [the nomograph] should be adequate for either normal or skewed crossings."

Therefore, for skewed crossings, the length of guide banks should be set using the nomograph for the side of the bridge crossing which yields the largest guide bank length.

Other Design Concerns: In some cases, where the cost of stone riprap facing is prohibitive, the guide bank can be covered with sod or other minimal protection. If this approach is selected, the design should allow for and stipulate the repair or replacement of the guide bank after each high water occurrence. Other measures which will minimize damage to approach embankments, and guide banks during high water are:

Keep trees as close to the toe of guide bank embankments as construction will permit. Trees will increase the resistance to flow near and around the toe of the embankment, thus reducing velocities and scour potential.

Do not allow the cutting of channels or the digging of borrow pits along the upstream side of approach

embankments and near guide banks. Such practices encourage flow concentration and increases velocities and erosion rates of the embankments.

In some cases, the area behind the guide bank may be too low to drain properly after a period of flooding. This can be a problem, especially when the guide bank is relatively impervious. Small drain pipes can be installed in the guide bank to drain this ponded water.

In some cases, only one approach will cut off the overbank flow. This is common when one of the banks is high and well defined. In these cases, only one guide bank may be necessary.

Design Example Of Guide Bank Installation

For the example design of a guide bank, Figure 1-3-39 will be used. This figure shows the cross-section of the channel and flood plain before the bridge is constructed and the plan view of the approach, guide banks, and embankments after the design steps outlined below are completed.

Step 1. Hydraulic Design Parameters

The first step in the design of guide banks requires the computation of the depth and velocity of the design flood in the main channel and in the adjacent overbank areas. These studies are performed by using step backwater computations upstream and through the bridge opening. The computer program HEC River Analysis System (RAS) is suitable for these computations. Using this program or by using conveyance curves developed from actual data, the discharges and depths in the channel and overbank areas can be determined. To use the conveyance curve approach, the designer is referred to example problem number 4 in Hydraulics of
Bridge Waterways (Bradley 1978) for methods to determine these discharges and areas. That publication also
contains another example of the design of a guide bank.

For this example, the total, overbank, and channel discharges, as well as the flow area are given. We also assume that a bridge will span a channel with a bottom width of 230 ft and that **the abutments will be set back 148 ft** from each bank of the main channel. The abutments of this bridge are spill-through with a side slope of 1V:2H. The design discharge is 12,360 cfs, which after backwater computations, results in a mean depth of 11.8 ft in the main channel and a mean channel velocity of 3 ft/s.

Step 2. Determine Q_f in the Left and Right Overbank

The depth in each overbank area is given as 3.9 ft and the widths of the left and right overbank areas are 295 ft
and 590 ft, respectively. Velocity in the overbank areas (assuming no railroad approach embankment, i.e., at an
upstream cross section) is 1.2 ft/s. The floodplain flow is equal to 1,413 cfs for the left overbank and 2,825 cfs for the right overbank.

Using the continuity equation and noting that the abutments are set back 148 ft from each bank, the flood plain discharge intercepted by each approach embankment is:

$Q = AV$

(Q_f) right = 2,825 - (148) (3.9) (1.2) = 2132 cfs

(Q_f) left = 1,413 - (148 (3.9) (1.2) = 720 cfs

Step 3. Determine Q_A and Q_f/Q_A for the Left and Right Overbank

The overbank discharge in the first 100 ft of opening adjacent to the left and right abutments needs to be determined next. Since for this case the flow is of uniform depth (3.9 ft) and velocity (1.2 ft/s) over the entire width of the floodplain, and both abutments are set back more than 100 ft from the main channel banks, the value of Q_A will be the same for both sides:

(Q_A) right = (100) (3.9) (1.2) = 468 cfs

(Q_A) left = (100 (3.9) (1.2) = 468 cfs

Figure 1-3-39. Example Guide Bank Design

For the left and right overbanks the reference values of Q_f / Q_A can be determined by simple division of the discharges determined in previous steps:

For design purposes, the largest value will result in the more conservative determination of the length of the guide banks, except where Step 4 indicates a guide bank is required for only one of the overbank areas.

Step 4. Determine the Length of the Guide Bank, L_s

The average channel velocity through the bridge opening can be determined by dividing the total discharge of the stream, Q, by the cross-sectional flow area at the bridge opening, A_{n2}, which in this case includes the main channel (2,714 ft^2) plus 148 ft of the left and right overbank areas adjacent to the abutments at the bridge opening (1,154 ft^2). Thus:

For Q_f / Q_A equal to 4.5 and an average channel velocity of 3.2 ft/s, the length of the guide bank is determined using the nomograph presented in Figure 1-3-38.

For the left abutment, a Q_f / Q_A of 1.5 and V_{n2} of 3.2 ft/s indicate that L_s would be less than 50 ft. Thus, no guide bank is required for the left overbank for this example.

Step 5. Miscellaneous Specifications

The offset of the guidebank is determined to be 55.2 ft by multiplying L_s by 0.4. The offset and length determine the plan layout of the guide bank. Coordinates of points along the centerline can be determined using EQ 26, which is the equation of an ellipse with a major to minor axis ratio of 2.5:1. The coordinates for a 138 ft long guide bank with a 55.2 ft offset are presented in Table 1-3-15.

These coordinates would be used for conceptual level design. For construction, coordinates at an offset or along
the toe of side slope would be necessary.

The crest of the guide bank must be a minimum of 2 ft above the design water surface (elevation 1070.2 ft). Therefore, the crest elevation for this example should be greater than or equal to 1072.2 ft. The crest width should be at least 3 ft. For this example, a crest width of 10 ft will be specified so that the guide bank can be easily constructed with dump trucks.

$$\left(\frac{Q_f}{Q_A}\right)_{right} = \frac{2132}{468} = 4.5$$

$$\left(\frac{Q_f}{Q_A}\right)_{left}$$

\)
/) |
= 720 =
468
15 .
V Q
n a
n
2
2
12360
118 230 2 3 9 148
= =
(.)( ) + (.)( )
V fts n2 = 3.2 /
(L)right ft s = 138

Table 1-3-15. Coordinates for Guide Bank on the Right Bank of Figure 10.4

Stone or rock riprap should be placed in the locations shown on Figure 1-3-39. This riprap should extend a minimum of 2 ft above the design water surface (elevation 1070.2 m) and below the intersection of the toe of the guide bank and the existing ground to the combined long-term degradation and contraction scour depth.

References for Guidebank Design

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Karaki, S.S., 1959. "Hydraulic Model Study of Spur Dikes for Highway Bridge Openings," Colorado State University, Civil Engineering Section, Report CER59SSK36, September, 47 pp.
Karaki, S.S., 1961. "Laboratory Study of Spur Dikes for Highway Bridge Protection," Highway Research Board Bulletin 286, Washington, D.C., p. 31.
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U.S. Army Corps of Engineers, 2001. "HEC-RAS River Analysis System," User's Manual, Version 3.0 , Hydrologic Engineering Center, Davis, CA.

3.6.4.4 Spurs

Background

A spur can be a pervious or impervious structure projecting from the streambank into the channel. Spurs are used to deflect flowing water away from, or to reduce flow velocities in critical zones near the streambank, to prevent erosion of the bank, and to establish a more desirable channel alignment or width. The main function of spurs is to reduce flow velocities near the bank, which in turn, encourages sediment deposition due to these
reduced velocities. Increased protection of banks can be achieved over time, as more sediment is deposited behind the spurs. Because of this, spurs may protect a streambank more effectively and at less cost than revetments. Furthermore, by moving the location of any scour away from the bank, partial failure of the spur can often be repaired before damage is done to structures along and across the stream.

Spurs are generally used to halt meander migration at a bend. They are also used to channelize wide, poorly defined streams into well-defined channels. The use of spurs to establish and maintain a well-defined channel

X (ft)	Y (ft)
0	55.2

30	53.9
60	49.7
90	41.8
120	27.3
138	0.0

location, cross section, and alignment in braided streams can decrease the required bridge lengths, thus decreasing the cost of bridge construction and maintenance.

Spur types are classified based upon their permeability as retarder spurs, retarder/deflector spurs, and deflector spurs. The permeability of spurs is defined simply as the percentage of the spur surface area facing the streamflow that is open. Deflector spurs are impermeable spurs which function by diverting the primary flow currents away from the bank. Retarder/deflector spurs are more permeable and function by retarding flow
velocities at the bank and diverting flow away from the bank. Retarder spurs are highly permeable and function by retarding flow velocities near the bank.

Design Considerations

Spur design includes setting the limits of bank protection required; selection of the spur type to be used; and design of the spur installation including spur length, orientation, permeability, height, profile, and spacing.

Longitudinal Extent of Spur Field. The longitudinal extent of channel bank requiring protection is discussed in Brown (1985, 1989). Figure 1-3-40 was developed from USACE studies of the extent of protection required at meander bends (USACE 1981). The minimum extent of bank protection determined from Figure 1-3-40 should be adjusted according to field inspections to determine the limits of active scour, channel surveys at low flow, and aerial photography and field investigations at high flow. Investigators of field installations of bank protection have found that protection commonly extends farther upstream than necessary and not far enough downstream. However, such protection may have been necessary at the time of installation. The lack of a sufficient length of protection downstream is generally more serious, and the downstream movement of meander bends should be considered in establishing the downstream extent of protection.

Spur Length. Spur length is taken here as the projected length of spur normal to the main flow direction or from the bank. Where the bank is irregular, spur lengths must be adjusted to provide for an even curvature of the thalweg. The length of both permeable and impermeable spurs relative to channel width affects local scour depth at the spur tip and the length of bank protected. Laboratory tests indicate that diminishing returns are realized from spur lengths greater than 20 percent of channel width. The length of bank protected measured in terms of

Figure 1-3-40. Extent of Protection Required at a Channel Bend (after USACE)

projected spur length is essentially constant up to spur lengths of 20 percent of channel width for permeable and impermeable spurs. Field installations of spurs have been successful with lengths from 3 to 30 percent of channel width. Impermeable spurs are usually installed with lengths of less than 20 percent while permeable spurs have been successful with lengths up to 25 percent of channel width. However, only the most permeable spurs were effective at greater lengths.

The above discussion assumes that stabilization of the bend is the only objective when spur lengths are selected. It also assumes that the opposite bank will not erode. Where flow constriction or changing the flow path is also an objective, spur lengths will depend on the degree of constriction required or the length of spur required to achieve the desired change in flow path. At some locations, channel excavation on the inside of the bend may be required where spurs would constrict the flow excessively. However, it may be acceptable to allow the stream to do its own excavation if it is located in uniformly graded sand

Spur Orientation. Spur orientation refers to spur alignment with respect to the direction of the main flow current in a channel. Figure 1-3-41 defines the spur angle such that an acute spur angle means that the spur is angled in an downstream direction and an angle greater than 90 indicates that the spur is oriented in a upstream direction. Permeable retarder spurs are usually designed to provide flow retardance near the streambank, and they perform

this function equally as well without respect to the spur angle. Since spurs oriented normal to the bank and projecting a given length into the channel are shorter than those at any other orientation, all retarder spurs should be constructed at 90 with the bank for reasons of economy.

Spur orientation at approximately 90 has the effect of forcing the main flow current (thalweg) farther from the concave bank than spurs oriented in an upstream or downstream direction. Therefore, more positive flow control is achieved with spurs oriented approximately normal to the channel bank. Spurs oriented in an upstream direction cause greater scour than if oriented normal to the bank, and spurs oriented in a downstream direction cause less scour.

It is recommended that the spur furthest upstream be angled downstream to provide a smoother transition of the flow lines near the bank and to minimize scour at the nose of the leading spur. Subsequent spurs downstream should all be set normal to the bank line to minimize construction costs.

Spur Permeability. The permeability of the spur depends on stream characteristics, the degree of flow retardance and velocity reduction required, and the severity of the channel bend. Impermeable spurs can be used on sharp bends to divert flow away from the outer bank. Where bends are mild and only small reductions in velocity are necessary, highly permeable retarder spurs can be used successfully. However, highly permeable spurs can also

Figure 1-3-41. Definition Sketch for Spur Angle (after Karaki 1959)

provide required bank protection under more severe conditions where vegetation and debris will reduce the permeability of the spur without destroying the spur. This is acceptable provided the bed load transport is high.

Spurs of varying permeability will provide protection against meander migration. Impermeable spurs provide more positive flow control but cause more scour at the toe of the spur and, when submerged, cause erosion of the streambank. High permeability spurs are suitable for use where only small reductions in flow velocities are necessary as on mild bends but can be used for more positive flow control where it can be assumed that clogging with small debris will occur and bed load transport is large. Spurs with permeability up to about 35 percent can be used in severe conditions but permeable spurs may be susceptible to damage from large debris and ice.

Spur Height and Crest Profile. Impermeable spurs are generally designed not to exceed the bank height because erosion at the end of the spur in the overbank area could increase the probability of outflanking at high stream stages. Where stream stages are greater than or equal to the bank height, impermeable spurs should be equal to the bank height. If flood stages are lower than the bank height, impermeable spurs should be designed so that overtopping will not occur at the bank. Bank erosion is more severe if the spur is oriented in the downstream direction.

The crest of impermeable spurs should slope downward away from the bank line, because it is difficult to construct and maintain a level spur of rock or gabions. Use of a sloping crest will avoid the possibility of overtopping at a low point in the spur profile, which could cause damage by particle erosion or damage to the streambank.

Permeable spurs, and in particular those constructed of light wire fence, should be designed to a height that will allow heavy debris to pass over the top. However, highly permeable spurs consisting of jacks or tetrahedrons are dependent on light debris collecting on the spur to make them less permeable. The crest profile of permeable spurs is generally level except where bank height requires the use of a sloping profile.

Bed and Bank Contact. The most common causes of spur failure are undermining and outflanking by the stream. These problems occur primarily in alluvial streams that experience wide fluctuations in the channel bed. Impermeable rock riprap spurs and gabion spurs can be designed to counter erosion at the toe by providing excess material on the streambed as illustrated in Figure 1-3-42 and Figure 1-3-43. As scour occurs, excess material is launched into the scour hole, thus protecting the end of the spur. Gabion spurs are not as flexible as riprap spurs and may fail in very dynamic alluvial streams.

Figure 1-3-42. Launching of Stone Protection on a Riprap Spur

(a) before launching at low flow, (b) during launching at high flow, and (c) after scour subsides (after Brown)

Figure 1-3-43. Gabion Spur Illustrating Flexible Mat Tip Protection
(a) before launching at low flow, (b) during launching at high flow, and (c) after scour subsides
(after Brown)

Permeable spurs can be similarly protected with a riprap toe as illustrated in Figure 1-3-44. The necessity for using riprap on the full length of the spur or any riprap at all is dependent on the erodibility of the streambed, the distance between the slats and the streambed, and the depth to which the piling are driven. This would also be appropriate as a retrofit measure at a spur that has been severely undermined, and as a design for locations at which severe erosion of the toe of the streambank is occurring.
Piles supporting permeable structures can also be protected against undermining by driving piling to depths below the estimated scour. Round piling are recommended because they minimize scour at their base. Extending the facing material of permeable spurs below the streambed also significantly reduces scour. If the retarder spur or retarder/deflector spur performs as designed, retardance and diversion of the flow within the
length of the structure may make it unnecessary to extend the facing material the full depth of anticipated scour except at the nose.
Spur Spacing. Spur spacing is a function of spur length, spur angle, permeability, and the degree of curvature of the bend. The flow expansion angle, or the angle at which flow expands toward the bank downstream of a spur, is a function of spur permeability and the ratio of spur length to channel width. This ratio is susceptible to alteration by excavation on the inside of the bend or by scour caused by the spur installation. Figure 1-3-45 indicates that the expansion angle for impermeable spurs is an almost constant 17. Spurs with 35 percent permeability have almost the same expansion angle except where the spur length is greater than about 18 percent of the channel width.
As permeability increases, the expansion angle increases, and as the length of spurs relative to channel width increases, the expansion angle increases exponentially. The expansion angle varies with the spur angle, but not
significantly.
Figure 1-3-44. Permeable Wood-slat Fence Spur Showing Launching of Stone Toe Material (after Brown)

Spur spacing in a bend can be established by first drawing an arc representing the desired flow alignment (Figure 1-3-46). This arc will represent the desired extreme location of the thalweg nearest the outside bank in
the bend. The desired flow alignment may differ from existing conditions or represent no change in conditions,
depending on whether there is a need to arrest erosion of the concave bank or reverse erosion that has already
occurred. If the need is to arrest erosion, permeable retarder spurs or retarder structures may be appropriate.
If the flow alignment must be altered in order to reverse erosion of the bank or to alter the flow alignment significantly, deflector spurs or retarder/deflector spurs are appropriate. The arc representing the desired flow

alignment may be a compound circular curve or any curve which forms a smooth transition in flow directions.

Figure 1-3-45. Relationship Between Spur Length and Expansion Angle for Several Spur Permeabilities (after Brown)

Figure 1-3-46. Spur Spacing in a Meander Bend (after Brown)

Next, draw an arc representing the desired bankline. This may approximately describe the existing concave bank or a new theoretical bankline which protects the existing bank from further erosion. Also, draw an arc connecting the nose (tip) of spurs in the installation. The distance from this arc to the arc describing the desired bank line, along with the expansion angle, fixes the spacing between spurs. The arc describing the ends of spurs projecting into the channel will be essentially concentric with the arc describing the desired flow alignment.

Establish the location of the spur at the downstream end of the installation. This is normally the protected abutment or guidebank at the bridge. Finally, establish the spacing between each of the remaining spurs in the installation (Figure 1-3-46). The distance between spurs, S, is the length of spur, L, between the arc describing the desired bank line and the nose of the spur multiplied by the cotangent of the flow expansion angle, . This length is the distance between the nose of spurs measured along a chord of the arc describing spur nose location. Remaining spurs in the installation will be at the same spacing if the arcs are concentric. The procedure is illustrated by Figure 1-3-46 and expressed in EQ 27.

$S = L \cot \theta$ **EQ 27**

where:

At less than bankfull flow rates, flow currents may approach the concave bank at angles greater than those estimated from Figure 1-3-45. Therefore, spurs should be well-anchored into the existing bank, especially the spur at the upstream end of the installation, to prevent outflanking.

Shape and Size of Spurs. In general, straight spurs should be used for most bank protection. Straight spurs are more easily installed and maintained and require less material. For permeable spurs, the width depends on the type of permeable spur being used. Less permeable retarder/deflector spurs which consist of a soil or sand embankment should be straight with a round nose as shown in Figure 1-3-47.

The top width of embankment spurs should be a minimum of 3 ft. However, in many cases the top width will be dictated by the width of any earth moving equipment used to construct the spur. In general a top width equal to the width of a dump truck can be used. The side slopes of the spur should be 1V:2H or flatter.

Riprap. Rock riprap should be placed on the upstream and downstream faces as well as on the nose of the spur to inhibit erosion of the spur. Depending on the embankment material being used, a gravel, sand, or geotextile may be required (see HEC-11). The designer is referred to HEC-11 and Article 3.6.4.5 for design procedures for sizing riprap at spurs.

It is recommended that riprap be extended below the bed elevation to a depth equal to the combined long-term degradation and contraction scour depth. Riprap should also extend to the crest of the spur, in cases where the spur would be submerged at design flow, or to 2 ft above the design flow, if the spur crest is higher than the design flow depth. Additional riprap should be placed around the nose of the spur (Figure 1-3-47), so that spur will be protected from scour. Figure 1-3-48 shows an example of an impermeable spur field and a close-up of a typical round nose spur installation.

S = spacing between spurs at the nose, m (ft)

L = effective length of spur, or the distance between arcs describing the toe of spurs and the

desired bank line, m (ft)

θ = expansion angle downstream of spur nose, degrees

Design Example of Spur Installation

Figure 1-3-49 illustrates a location at which a migrating bend threatens an existing bridge (existing conditions
are shown with a solid line). Ultimately, based upon the following design example, seven spurs will be required.
Although the number of spurs is not known in advance, the spurs (and other design steps) are shown as dashed
lines on Figure 1-3-49 as they will be specified after completing the following design example. Assume that the
width of the river from the desired (north) bankline to the existing (south) bankline is 164 ft.
For this example, it is desirable to establish a different flow alignment and to reverse erosion of the concave (outside) bank. The spur installation has two objectives: (1) to stop migration of the meander before it damages the railroad stream crossing, and (2) to reduce scour at the bridge abutment and piers by aligning flow
in the channel with the bridge opening. Impermeable deflector spurs are suitable to accomplish these objectives and the stream regime is favorable for the use of this type of countermeasure. The expansion angle for this spur type is approximately 17 for a spur length of about 20 percent of the desired channel width, as indicated in Figure 1-3-45.

Step 1. Sketch Desired Thalweg

The first step is to sketch the desired thalweg location (flow alignment) with a smooth transition from the upstream flow direction through the curve to an approach straight through the bridge waterway (Figure 1-3-49). Visualize both the high-flow and low-flow thalwegs. For an actual location, it would be necessary to examine a greater length of stream to establish the most desirable flow alignment. Then draw an arc representing the desired bankline in relation to thalweg locations. The theoretical or desired left bank line is established as a continuation of the bridge abutment and left bank downstream through the curve, smoothly joining the left bank at the upstream extremity of eroded bank.

Step 2. Sketch Alignment of Spur Tips

The second step is to sketch a smooth curve through the nose (tip) locations of the spurs, concentric with the desired bankline alignment. Using a guideline of 20 percent of the desired channel width for impermeable spurs (see Spur Length) the distance, L, from the desired bankline to the spur tips (Figure 1-3-49) would be:
L = 0.20(164ft) = 33ft

Figure 1-3-47. Typical Straight, Round Nose Spur

Figure 1-3-48. Impermeable Spur Field in Top Photograph With Close-up Shot of One Spur in the Lower Photograph, Vicinity of the Richardson Highway, Delta River, Alaska

Step 3. Locate First Spur

Step number three is to locate spur number 1 so that flow expansion from the nose of the spur will intersect the streambank downstream of the abutment. This is accomplished by projecting an angle of 17 from the abutment

alignment to an intersection with the arc describing the nose of spurs in the installation or by use of EQ 27. Spurs are set at 90 to a tangent with the arc for economy of construction. Alternatively, the first spur could be considered to be either the upstream end of the abutment or guide bank if the spur field is being installed upstream of a bridge. Thus, the spur spacing, S, would be:

$S = L \cot \theta = (33ft) \cot 17^{\circ} = 108ft$

It may be desirable to place riprap on the streambank at the abutment. Furthermore, the size of the scour hole at the spur directly upstream of the bridge should be estimated using the procedures described in Article 3.5.5. If the extent of scour at this spur overlaps local scour at the pier, total scour depth at the pier may be increased. This can be determined by extending the maximum scour depth at the spur tip, up to the existing bed elevation at the pier at the angle of repose.

Step 4. Locate Remaining Spurs

Spurs upstream of spur number 1 are then located by use of EQ 27, using dimensions as illustrated in Figure 1-3-46 (i.e., the spacing, S, determined in Step 3). Using this spur spacing, deposition will be encouraged between the desired bank line and the existing eroded bank.

The seventh and last spur upstream is shown oriented in a downstream direction to provide a smooth transition of the flow approaching the spur field. This spur could have been oriented normal to the existing bank, and been shorter and more economical, but might have caused excessive local scour. Orienting the furthest upstream spur at an angle in the downstream direction provides a smoother transition into the spur field, and decreases scour at the nose of the spur. As an alternative, a hard point could be installed where the bank is beginning to erode (see Article 3.6.4.9). In this case the hard point can be considered as a very short spur which is located at the intersection of the actual and planned bank lines. In either case, spurs or hard points should be anchored well into the bank to prevent outflanking.

Figure 1-3-49. Example of Spur Design

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3.6.4.5 Revetments

Introduction

Revetments are used to provide protection for embankments, streambanks, and streambeds. They may be flexible or rigid and can be used to counter all erosion mechanisms. They do not significantly constrict channels or alter flow patterns. Revetments do not provide resistance against slumping in saturated streambanks and embankments, and are relatively unsuccessful in stabilizing streambanks and streambeds in degrading streams. Special precautions must be observed in the design of revetments for degrading channels.

Flexible Revetments

Flexible revetments include rock riprap, rock-and-wire mattresses, gabions, precast concrete blocks, rock-fill trenches, windrow revetments, used tire revetments, and vegetation. Rock riprap adjusts to distortions and local displacement of materials without complete failure of the revetment installation. However, flexible rock-and-wire mattress and gabions may sometimes span the displacement of underlying materials, but usually
can adjust to most local distortions (see Article 3.6.4.6). Used tire mattresses and precast concrete block mattresses are generally stiffer than rock riprap and gabions and, therefore, do not adjust well to local displacement of underlying materials.

Design guidelines, design procedures, and suggested specifications for rock riprap, wire enclosed rock, stacked
block gabions, and precast concrete blocks are included in HEC-11. Since rock riprap is commonly used as a countermeasure for stream bank erosion, a short discussion of the types of rock riprap and a design procedure
as discussed in HEC-11 follows.

Riprap as discussed in this section is defined as a flexible channel or bank lining consisting of a well-graded mixture of angular rock usually dumped in place. Other types of riprap are "hand-placed" and "keyed or plated"
riprap. Hand-placed riprap is carefully placed by hand or by a mechanized manner in a definite pattern with voids between the large stone being filled with smaller rock. Plated riprap is placed on the bank with a skip and
tamped into place using a heavy steel plate leaving a smoother surface than dumped riprap. See HEC-11 for more information on each of these types.

Dumped riprap does not mean end dumping from trucks and allowing the material to roll down the slope which
can cause size segregation. It means that the riprap is placed in a manner to prevent segregation by using a crane with a bucket or dragline. Regardless of how it is placed, care should be taken to prevent segregation of the rock mixture. Dumped riprap should form a layer of loose stone where individual stones may move independently to adjust to the movement of the bank material being protected. This minor movement may occur without complete failure of the installation. This movement allows the riprap to be somewhat "self healing" and is one of the main advantages of dumped rock riprap.

Design Guidelines

HEC-11 provides design guidance for sizing the rock for dumped riprap used for bank protection. The procedure is based on the tractive force theory but has velocity as its primary design parameter. The equation is based on the assumption of uniform or gradually varying flow. A stability factor is used to correct the equation for bends and turbulent mixing at rapidly varying flow conditions.

The stone size is established by this equation:

where:

where:

D_{50} = median particle size, m (ft)

C = correction for specific gravity and stability factor

V_a = average velocity in the main channel, m/s (fps)

d_{avg} = average flow depth in the main flow channel, m (ft)

K_1 = bank angle correction factor as given below

K_u = 0.0059 SI

K_u = 0.001 English

θ = bank angle with the horizontal

ϕ = riprap material’s angle of repose as given in Figure 1-3-50

D K C V

d K

u a

avg

50

3

0 5

1

15

=

. . **EQ 28**

K_1 1 **EQ 29** sin θ 2

sin ϕ 2

– --------------

0.5

=

The average flow depth and velocity used in EQ 28 are main channel values where the main channel is defined
as the area between the channel banks.

The correction for the specific gravity and the stability factor is defined by the following equation:

Figure 1-3-50. Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone (Chen and Cotton 1988)

C **EQ 30** $1.61(SF)^{1.5}$

$(S_s - 1)^{1.5}$

= -------------------------------

where:

The stability factor (SF) is defined as the ratio of the riprap material critical shear stress and average tractive force exerted by the flow field. As long as the SF is greater than 1, the critical shear stress of the material is greater than the flow induced tractive stress, and the riprap is considered stable. A SF of 1.2 was used in the development of EQ 28.

The SF may be used to reflect the level of uncertainty in the conditions at the site due to discharge estimation inaccuracies, debris, ice impacts, etc. Suggested values for the SF are:

Thickness of Riprap. All stones should be contained reasonably well within the riprap layer thickness. The following criteria are given in HEC-11.

- It should not be less than the spherical diameter of the D_{100} stone or less than 1.5 times the spherical diameter of the D_{50} stone, whichever results in the greater thickness.
- It should not be less than 1 ft for practical placement.
- The thickness determined by either 1 or 2 should be increased by 50 percent when the riprap is placed underwater to compensate for uncertainties associated with this placement.
- An increase in layer thickness of 0.5 to 1 ft, accompanied by an increase in stone sizes, should be made where the riprap will be subject to attack by floating debris, ice, or by waves from boat wakes, wind, or bedforms.

Gradation of Riprap. The gradation of stones in riprap revetment affects the riprap's resistance to erosion. The

stone should be reasonably well graded throughout the riprap layer thickness. Specifications should provide for
two limiting gradation curves, and the stone gradation (as determined from a field test sample) should lay within these limits. The gradation limits should not be so restrictive that production costs would be excessive. HEC-11 presents suggested guidelines for establishing gradation limits (see Table 1-3-16). Table 1-3-16 and Table 1-3-17 present six suggested gradation classes based on AASHTO specifications (AASHTO 1999).

S_s = specific gravity of the rock riprap

SF = stability factor as described below

Condition	SF Range
Uniform flow conditions: Straight or mildly curving reach (curve radius/channel width >30); impact from wave action and floating debris is minimal; little or no uncertainty in design parameters.	1.0 - 1.2
Gradually varying flow: Moderate bend curvature (30 > curve radius/channel width >10): impact from waves or floating debris moderate.	1.3 - 1.6
Approaching rapidly varying flow: Sharp bend curvature (10 > curve radius/channel width); significant impact potential from floating debris and/or ice; significant wind and/or boat generated waves (1 - 2 ft); high flow turbulence; turbulent mixing at bridge abutments; significant uncertainty in design parameters.	1.6 - 2.0

Table 1-3-16. Rock Riprap Gradation Limits

Table 1-3-17. Riprap Gradation Classes (English)

Gradation of the riprap being placed is controlled by visual inspection. To aid the inspector's judgment, two or
more samples of riprap of the specified gradation should be prepared by sorting, weighing, and remixing in proper proportions. Each sample should weigh about 5 to 10 tons. One sample should be placed at the quarry and one sample at the construction site. The sample at the construction site could be part of the finished riprap
blanket. These samples should be used as a frequent reference for judging the gradation of the riprap supplied.

Filter Systems. A filter system should be provided to prevent the migration of the fine soil between the voids of
the riprap. The system may be either a granular filter or an engineering filter fabric. Consultation with a geotechnical engineer may be useful in making the proper selection.

Stone Size Range m (ft)	Stone Weight Range kg (lb)	Percent of Gradation Smaller Than
1.5 D_{50} to 1.7 D_{50}	3.0 W_{50} to 5.0 W_{50}	100
1.2 D_{50} to 1.4 D_{50}	2.0 W_{50} to 2.75 W_{50}	85
1.0 D_{50} to 1.15 D_{50}	1.0 W_{50} to 1.5 W_{50}	50
0.4 D_{50} to 0.6 D_{50}	0.1 W_{50} to 0.2 W_{50}	15

Riprap
Class
Rock Size[1]
(ft)

Rock Size[2]
(lbs)
Percent of Riprap
Smaller Than
Facing 1.30
0.95
0.40
200
75
5
100
50
10
Light 1.80
1.30
0.40
500
200
5
100
50
10
1/4 Ton 2.25
1.80
0.95
1,000
500
75
100
50
10
1/2 Ton 2.85
2.25
1.80
2,000
1,000
500
100
50
5
1 Ton 3.60
2.85
2.25
4,000
2,000
1,000
100
50
5
2 Ton 4.50
3.60
2.85
8,000
4,000
2,000

100
50
5
1Assuming a specific gravity of 2.65
2Based on AASHTO gradations

Granular Filters. In using a granular filter system, the filter ratio as stated in the following relationships should be met.
The left side of the inequality in EQ 31 is intended to prevent erosion (piping) through the filter and the center portion provides for adequate permeability for structural bedding. The right portion provides a uniformity criterion.
If a single layer of filter will not satisfy the equation, two or more layers must be used. The filter requirement applies between the bank material and the filter as well as the filter and the riprap. The thickness of the filter blanket should be from 150 mm (6 in) and 380 mm (15 in) for a single layer, or from 100 mm (4 in) to 200 mm (8 in) for individual layers of a multilayer installation.
Engineering Fabric Filters. For the proper design of a geotextile filter system, see Holtz et al. (FHWA HI-95-038). The fabric should provide drainage and filtration. Therefore, both functions should be considered in the selection of the filter material.
Edge Treatment. To prevent undermining at the toe and flanks of the riprap, special edge treatment may be required such as:

- Extending the lower toe of the riprap below the anticipated contraction scour and long-term degradation depth.
- Placing launchable stone at the toe of the installation that will slide into the scour hole as it develops. This method requires extra material to be placed at bottom of the installation in a trench or extending into the stream (Figure 1-3-51 and Figure 1-3-52). For additional information, see HEC-11.
- The flanks may be protected as illustrated in Figure 1-3-53. In Section A-A, the area shown as "compacted backfill" may be completely filled with riprap.

D coarser layer
D finer layer
D coarser layer
D finer layer
15
85
5 15
15
( ) 40
( )
( )
( )
< < < **EQ 31**

Revetment Riprap Design Example

The following design example illustrates the general revetment riprap design procedure. From a field survey of

the site and an analysis of the stream using a water surface profile program such as HEC-RAS the following data have been established.

Given:

Channel width = 300 ft

Bend radius = 200 ft

Average velocity in main channel (V_a) = 12.6 fps

Average depth in main channel (d_a) = 12 ft

Available rock riprap has a specific gravity of 2.60 and is considered angular.

A 1 vertical to 2 horizontal (1V:2H) bank slope is to be used.

Figure 1-3-51. Methods of Providing Toe Protection (USACE 1991)

Figure 1-3-52. Alternative Method of Providing Toe Protection (HEC-11)

Solution:

Using EQ 28, EQ 29, and EQ 30, the following size is established.

Figure 1-3-53. Flank Details (HEC-11)

From Figure 1-3-46 for angular stone, a value of 41° for the angle of repose would be a good initial estimate to
use. For a side slope of 1V:2H:

Assuming for a gradually varying flow with moderate bend curvature, the stability factor (SF) is 1.6. (See the previous guidance for stability factor.)

The required stone size is then found.

Using this stone size of 1.5 ft, recheck the angle of repose. It would be close to the original 41° that was assumed and would be acceptable.

Taking this computed size of stone, compare it to a class of riprap that is available and use the next larger size (perhaps the AASHTO 1/4 ton class riprap).

The layer thickness would be twice the mean size (2 D_{50}) or the thickness equal to the D_{100}.

The need for a filter system depends on the parent material at the site. Normally a filter system will be required. It may be either a granular filter or a geotextile.

Rock-Fill Trenches and Windrow Revetment

Rock-fill trenches are structures used to protect banks from caving caused by erosion at the toe. A trench is excavated along the toe of the bank and filled with rocks as shown in Figure 1-3-54. The size of trench to hold the rock fill depends on expected depths of scour.

D_{50} **EQ 32**

$K_u C V_a$

3

d_{avg}

$0.5 K_1$

1.5

= --------------------------------- K_1 1 sin θ 2

sin ϕ 2

– --------------

0.5

= C 1.61 SF ( )1.5

(Ss – 1)1.5

= -------------------------------

sin θ = 1 = . sin ϕ = sin ° = .

5

0 447 41 0 656

K1 1 sin θ 2

sin ϕ 2

– --------------

0.5

1 (0.447)2

(0.656)2

– ---------------------

0.5

= = = 0.73

C 1.61(SF)1.5

(Ss – 1)1.5

------------------------------- 1.61(1.6)1.5

(2.60 – 1)1.5

= = -------------------------------- = 1.61

1.5 ft

(12) (0.73)

(0.001)(1.61)12.6)

d K

K CV

D 0.5 1.5

3

5 . 11

0.5

avg

3

u a

50 = = =

As the streambed adjacent to the toe is eroded, the toe trench is undermined and the rock fill slides downward to pave the bank. It is advantageous to grade the banks before placing riprap on the slope and in the toe trench.

The slope should be at such an angle that the saturated bank is stable while the stream stage is falling.

An alternative to a rock-fill trench at the toe of the bank is to excavate a trench above the water line along the top of the bank and fill the trench with rocks. As the bank erodes, stone material in the trench is added on an as-needed basis until equilibrium is established. This method is applicable in areas of rapidly eroding banks of medium to large size streams.

Windrow revetment (Figure 1-3-55) consists of a supply of rock deposited along an existing bank line at a location beyond which additional erosion is to be prevented. When bank erosion reaches and undercuts the

supply of rock, it falls onto the eroding area, thus giving protection against further undercutting. The resulting bank line remains in a near natural state with an irregular appearance due to intermittent lateral erosion in the windrow location. The treatment particularly lends itself to the protection of adjacent wooded areas, or placement along stretches of presently eroding, irregular bank line.

The effect of windrow revetment on the interchange of flow between the channel and overbank areas and flood flow distribution in the flood plain should be carefully evaluated. Windrow installations will perform as guide banks or levees and may adversely affect flow distribution at bridges or cause local scour. Tying the windrow to the embankment at an abutment would be contrary to the purpose of the windrow since the rock is intended to fall into the channel as the bank erodes. This would potentially expose the abutment.

The following observations and conclusions from model investigations of windrow revetments and rock-fill trenches may be used as design guidance. More definitive guidance is not presently available (USACE 1981).

• The application rate of stone is a function of channel depth, bank height, material size, and estimated bed scour.

• A triangular windrow is the least desirable shape, a trapezoidal shape provides a uniform blanket of rock on an eroding bank, and a rectangular shape provides the best coverage. A rectangular shape is most easily placed in an excavated trench.

Figure 1-3-54. Rock-fill Trench (after HDS 6)

• Bank height does not significantly affect the final revetment; however, high banks tend to produce a nonuniform revetment alignment. Large segments of bank tend to break loose and rotate slightly on high banks, whereas low banks simply "melt" or slough into the stream.

Figure 1-3-55. Windrow Revetment, Definition Sketch (after USACE 1981)

• Stone size influences the thickness of the final revetment, and a smaller gradation of stone forms a more dense, closely chinked protective layer. Stones must be large enough to resist being transported by the stream, and a well-graded stone should be used to ensure that the revetment does not fail from leaching of the underlying bank material. Large stone sizes require more material than smaller stone sizes to produce the same relative thickness of revetment. In general, the greater the stream velocity, the steeper the side slope of the final revetment. The final revetment slope will be about 15 percent flatter than the initial bank slope.

• A windrow segment should be extended landward from the upstream end to reduce the possibility of outflanking of the windrow.

Rigid Revetments

Rigid revetments are generally smoother than flexible revetments and thus improve hydraulic efficiency and are generally highly resistant to erosion and impact damage. They are susceptible to damage from the removal of foundation support by subsidence, undermining, hydrostatic pressures, slides, and erosion at the perimeter.

They are also among the most expensive streambank protection countermeasures.

Concrete Pavement

Concrete paving should be used only where the toe can be adequately protected from undermining and where hydrostatic pressures behind the paving will not cause failure. This might include impermeable bank materials and portions of banks which are continuously under water. Sections intermittently above water should be

provided with weep holes. Refer to HEC-11 for design of concrete pavement revetment.

Sacks

Burlap sacks filled with soil or sand-cement mixtures have long been used for emergency work along levees and
streambanks during floods (Figure 1-3-56). Commercially manufactured sacks (burlap, paper, plastics, etc.) have been used to protect streambanks in areas where riprap of suitable size and quality is not available at a reasonable cost. Sacks filled with sand-cement mixtures can provide long-term protection if the mixture has set
up properly, even though most types of sacks are easily damaged and will eventually deteriorate. Sand-cement
sack revetment construction is not economically competitive in areas where good stone is available. However,
where quality riprap must be transported over long distances, sack revetment can often be placed at a lesser cost than riprap.

If a sack revetment is to be constructed, the sacks should be filled with a mixture of 15 percent cement (minimum) and 85 percent dry sand (by weight). The filled sacks should be placed in horizontal rows like common house brick beginning at an elevation below any toe scour (alternatively, riprap can be placed at the toe to prevent undermining of the bank slope). The successive rows should be stepped back approximately onehalf-
bag width to a height on the bank above which no protection is needed. The slope of the completed revetment should not be steeper than 1:1. After the sacks have been placed on the bank, they can be wetted down for a quick set or the sand-cement mixture can be allowed to set up naturally through rainfall, seepage or
condensation. If cement leaches through the sack material, a bond will form between the sacks and prevent free drainage. For this reason, weepholes should be included in the revetment design. The installation of weepholes will allow drainage of groundwater from behind the revetment thus helping to prevent a pressure buildup that could cause revetment failure. This revetment requires the same types of toe protection as other types of rigid revetment.

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Figure 1-3-56. Typical Sand-cement Bag Revetment (after Brown 1985)

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3.6.4.6 Wire-Enclosed Rock

Wire-enclosed rock (gabion) revetments consist of rectangular wire mesh baskets filled with rock. The most common types of wire-enclosed revetments are mattresses and stacked blocks. The wire cages which make up
the mattresses and gabions are available from commercial manufacturers. If desired, the wire baskets can also
be fabricated from available wire fencing materials. This section provides design guidance for stacked block gabion revetment. Reference to HEC-11 is suggested for design guidance on gabion mattresses.

As a revetment, wire-enclosed rock has limited flexibility. They will flex with bank surface subsidence; however, if excessive subsidence occurs, the baskets will span the void until the stresses in rock-filled baskets exceed the tensile strength of wire strands. At this point, the baskets will fail (Escarameia 1998).

The conditions under which wire-enclosed rock is applicable are similar to those of other revetments. However,
their economic use is limited to locations where the only rock available economically is too small for use as rock
riprap slope protection. The primary advantages of wire-enclosed rock revetments include:

Their ability to span minor pockets of bank subsidence without failure
The ability to use smaller, lower quality, and less dense, rock in the baskets

Disadvantages of the use of wire-enclosed rock revetments include:

Susceptibility of the wire baskets to corrosion and abrasion damage
High labor costs associated with fabricating and filling the wire baskets
More difficult and expensive repair than standard rock protection
Less flexibility than standard rock protection

The most common failure mechanism of wire basket revetments has been observed to be failure of the wire baskets. Failure from abrasion and corrosion of the wire strands has even been found to be a common problem
when the wire is coated with plastic. The plastic coating is often stripped away by abrasion from sand, gravel, cobbles, or other sediments carried in natural stream flows (particularly at and near flood stages). Once the wire has been broken, the rock in the baskets is usually washed away. To avoid the problem of abrasion and

corrosion of the wire baskets, it is recommended that wire-enclosed rock revetments not be used on lower portions of the channel bank in environments subject to significant abrasion or corrosion.

An additional failure mechanism has been observed when the wire basket units are used in high-velocity, steepslope
environments. Under these conditions, the rock within individual baskets shifts downstream, deforming the baskets as the material moves. The movement of material within individual baskets will sometimes result in exposure of filter or base material. Subsequent erosion of the exposed base material can cause failure of the revetment system.

Stacked Block Gabions

Stacked block gabion revetments consist of rectangular wire baskets which are filled with stone and stacked in

a stepped-back fashion to form the revetment surface (Figure 1-3-57). They are also commonly used at the toe
of embankment slopes as toe walls which help to support other upper bank revetments and prevent undermining.
As illustrated in Figure 1-3-57, the rectangular basket or gabion units used for stacked configurations are more
equidimensional than those typically used for mattress designs. That is, they typically have a square cross section. Commercially available gabions used in stacked configurations include those listed in Table 1-3-18 having 3-foot widths and thickness. Other commercially available sizes can also be used in the stacked block configurations.
Conceptually, the gabion units for stacked block configurations could also be fabricated from available fencing materials. However, the labor intensive nature of such an installation makes it impractical in most cases. Therefore, only commercially available units are considered in the following design guidelines.

Design Guidelines for Stacked Block Gabions

Components of stacked gabion revetment design include layout of a general scheme or concept, bank and foundation preparation, unit size and configuration, stone size and quality, edge treatment, backfill and filter considerations, and basket or rock enclosure fabrication. Design guidelines for stone size and quality, and bank
preparation are generally available from manufacturer's literature (see also HEC-11).
General: Stacked gabion revetments are typically used when the slope to be protected is greater than 1:1 or when the purpose of the revetment is for flow training. They can also be used as retaining structures when space limitations prohibit bank grading to a slope suitable for other revetments. Typical design schemes include flow training walls, Figure 1-3-57(a), and low or high retaining walls (Figure 1-3-57(b) and (c), respectively.
Stacked gabion revetments must be based on a firm foundation. The foundation or base elevation of the structure should be well below any anticipated scour depth. Additionally, in alluvial streams where channel bed
fluctuations are common, an apron should be used as illustrated in Figure 1-3-57(a) and (b). Aprons are also recommended for situations where the estimated scour depth is uncertain.

Table 1-3-18. Standard Gabion Sizes

Thickness (ft)	Width (ft)	Length (ft)	Wire-Mesh Opening Size (in x in)
0.75	6	9	2.5 x 3.25
0.75	6	12	2.5 x 3.25
1.	3	6	3.25 x 4.5
1.	3	9	3.25 x 4.5
1.	3	12	3.25 x 4.5
1.5	3	6	3.25 x 4.5
1.5	3	9	3.25 x 4.5
1.5	3	12	3.25 x 4.5
3	3	6	3.25 x 4.5
3	3	9	3.25 x 4.5
3	3	12	3.25 x 4.5

Figure 1-3-57. Typical Stacked Block Gabion Revetment Details

(a) training well with counterforts, (b) stepped back low retaining wall with apron; (c) high retaining

wall, stepped-back configuration; (d) high retaining wall, batter type

Size and Configuration: Common commercial sizes for stacked gabions are listed in Table 1-3-18. The most common sizes used are those having widths and depths of 3 ft. Sizes less than 1 ft thick are not practical for stacked gabion installations.

Typical design configurations include flow training walls and structural retaining walls. The primary function of flow training walls (Figure 1-3-57(a)) is to establish normal channel boundaries in rivers where erosion has created a wide channel, or to realign the river when it is encroaching on an existing or proposed structure. A stepped-back wall is constructed at the desired bank location and counterforts are installed to tie the walls to the channel bank at regular intervals as illustrated. The counterforts are installed to form a structural tie between the training wall and the natural stream bank, and to prevent overflow from scouring a channel behind the wall. Counterforts should be spaced to eliminate the development of eddy or other flow currents between the training wall and the bank which could cause further erosion of the bank. The dead water zones created by the counterforts so spaced will encourage sediment deposition behind the wall which will enhance the stabilizing characteristics of the wall.

Retaining walls can be designed in either a stepped-back configuration as illustrated in Figure 1-3-57(b) and (c), or a batter configuration as illustrated in Figure 1-3-57(d). Structural details and configurations can vary from site to site.

Gabion walls are gravity structures and their design follows standard engineering practice for retaining structures. Design procedures are available in standard soil mechanics texts as well as in gabion manufacturer's literature.

Edge Treatment: The flanks and toe of stacked block gabion revetments require special attention. The upstream and downstream flanks of these revetments should include counterforts, see Figure 1-3-57(a). The counterforts should be placed 12 to 18 ft from the upstream and downstream limits of the structure, and should extend a minimum of 12 ft into the bank.

The toe of the revetment should be protected by placing the base of the gabion wall at a depth below anticipated scour depths. In areas where it is difficult to predict the depth of expected scour, or where channel bed fluctuations are common, it is recommended that a mattress apron be used. The minimum apron length should be equal to 1.5 times the anticipated scour depth below the apron. This length can be increased in proportion to the level of uncertainty in predicting the local toe scour depth.

Backfill/Filter Requirements: Standard retaining wall design requires the use of selected backfill behind the retaining structure to provide for drainage of the soil mass behind the wall. The permeable nature of gabion structures permits natural drainage of the supported embankment. However, since material leaching through the gabion wall can become trapped and cause plugging, it is recommended that a granular backfill material be used, see Figure 1-3-57(d). The backfill should consist of a 2- to 12-inch (5.1- to 30.5-cm) layer of graded crushed stone backed by a layer of fine granular backfill.

Basket Fabrication: Commercially fabricated basket units are formed from galvanized steel wire mesh of triple twist hexagonal weave. The netting wire and binding wire specifications are the same discussed for mattress units. Specifications for galvanizing and PVC coatings are also the same for block designs as for mattresses. Figure 1-3-58 illustrates typical details of basket fabrication.

Construction

Construction details for gabion installations typically vary with the design and purpose for which the protection is being provided. Several typical design schematics are presented in Figure 1-3-57 and Figure 1-3-58.

As with mattress designs, fabrication and filling of individual basket units can be done at the site, or at an offsite location. The most common practice is to fabricate and fill individual gabions at the design site. The following steps outline the typical sequence used for installing a stacked gabion revetment or wall:

Step 1. Prepare the revetment foundation. This includes excavation for the foundation and revetment wall.

Step 2. Place the filter and gabion mattress (for designs which incorporate this component) on the prepared grade, then sequentially stack the gabion baskets to form the revetment system.

Step 3. Each basket is unfolded and assembled by lacing the edges together and the diaphragms to the sides.

Step 4. Fill the gabions to a depth of 1-foot with stone form 4 to 12 inches in diameter. Place one connecting wire in each direction and loop it around two meshes of the gabion wall. Repeat this operation until the gabion is filled.

Step 5. Wire adjoining gabions together by their vertical edges; stack empty gabions on the filled gabions and wire them at front and back.

Step 6. After the gabion is filled, fold the top shut and wire it to the ends, sides and diaphragms.

Figure 1-3-58. Gabion Basket Fabrication

Step 7. Crushed stone and granular backfill should be placed in intervals to help support the wall structure. It is recommended that backfill be placed at 3-course intervals.

References for Wire Enclosed Rock

Brown, S.A. and Clyde, E.S., 1989. "Design of Riprap Revetment," Hydraulic Engineering Circular No.11, FHWA-IP-89-016. Prepared for the Federal Highway Administration, Washington, D.C.

Escarameia, M., 1998. "River and Channel Revetments – A Design Manual," Thomas Telford Limited, London.

3.6.4.7 Check Dams/Drop Structures

Background

Check dams or channel drop structures are used downstream of bridge crossings to arrest head cutting and maintain a stable streambed elevation in the vicinity of the bridge. Check dams are usually built of rock riprap, concrete, sheet piles, gabions, or treated timber piles. The material used to construct the structure depends on the availability of materials, the height of drop required, and the width of the channel. Rock riprap and timber pile construction have been most successful on channels having small drops and widths less than 100 ft. Sheet piles, gabions, and concrete structures are generally used for larger drops on channels with widths ranging up to 300 ft. Check dam location with respect to the bridge depends on the hydraulics of the bridge reach and the amount of headcutting or degradation anticipated.

Check dams can initiate erosion of banks and the channel bed downstream of the structure as a result of energy dissipation and turbulence at the drop. This local scour can undermine the check dam and cause failure. The use of energy dissipators downstream of check dams can reduce the energy available to erode the channel bed and banks. **In some cases it may be better to construct several consecutive drops of shorter height to minimize erosion.** Concrete lined basins as discussed later may also be used.

Lateral erosion of channel banks just downstream of drop structures is another adverse result of check dams and is caused by turbulence produced by energy dissipation at the drop, bank slumping from local channel bed
erosion, or eddy action at the banks. Bank erosion downstream of check dams can lead to erosion of bridge approach embankments and abutment foundations if lateral bank erosion causes the formation of flow channels around the ends of check dams. The usual solution to these problems is to place riprap revetment on the streambank adjacent to the check dam (see Article 3.6.4.5).

Erosion of the streambed can also be reduced by placing rock riprap in a preformed scour hole downstream of
the drop structure. A row of sheet piling with top set at or below streambed elevation can keep the riprap from
moving downstream. Because of the problems associated with check dams, the design of these countermeasures requires designing the check dams to resist scour by providing for dissipation of excess energy
and protection of areas of the bed and the bank which are susceptible to erosive forces.

Bed Scour For Vertical Drop Structures

Estimating Bed Scour. The most conservative estimate of scour downstream of channel drop structures is for vertical drops with unsubmerged flow conditions. For the purposes of design the maximum expected scour can
be assumed to be equal to the scour for a vertical, unsubmerged drop, regardless of whether the drop is actually
sloped or is submerged.

A sketch of a typical vertical drop structure with a free overfall is shown in Figure 1-3-59. An equation developed by the Bureau of Reclamation (USBR) is recommended to estimate the depth of scour downstream of
a vertical drop:

where:

It should be noted that H_t is the difference in the total head from upstream to downstream. This can be computed using the energy equation for steady uniform flow:

d_s = local scour depth for a free overall, measured from the streambed downstream of the drop, m (ft)

q = discharge per unit width, $m^3/s/m$ (cfs/ft)

H_t = total drop in head, measured from the upstream to the downstream energy grade line, m (ft)

d_m, Y_d = tailwater depth, m (ft)

K_u = 1.90 (SI)

K_u = 1.32 (English)

Figure 1-3-59. Schematic of a Vertical Drop Caused by a Check Dam

d **EQ 33** s KuHt
0.225q0.54 = – dm

H Y V
g
Z Y V
g
Z t u
u
u d
d
d = + +

)
) | − +
+
(
(|)
) |
2 2
2 2
EQ 34

where:
The subscripts u and d refer to upstream and downstream of the channel drop, respectively.
The depth of scour as estimated by the above equation is independent of the grain size of the bed material. This
concept acknowledges that the bed will scour regardless of the type of material composing the bed, but the rate
of scour depends on the composition of the bed. In some cases, with large or resistant material, it may take years or decades to develop the maximum scour hole. In these cases, the design life of the bridge may need to be considered when designing the check dam.
The check dam must be designed structurally to withstand the forces of water and soil assuming that the scour hole is as deep as estimated using the equation above. Therefore, the designer should consult geotechnical and structural engineers so that the drop structure will be stable under the full scour condition. In
some cases, a series of drops may be employed to minimize drop height and construction costs of foundations.
Riprap or energy dissipation could be provided to limit depth of scour (see, for example, Peterka and HEC-14).

Check Dam Design Example

Given:

Channel degradation is threatening bridge foundations. Increasing the bed elevation 4.6 ft will stabilize the channel at the original bed level. A drop structure will raise the channel bed and reduce upstream channel slopes, resulting in greater flow depths and reduced velocity upstream of the structure. For this example, as illustrated by Figure 1-3-60, the following hydraulic parameters are used:

Solution:

H_t is calculated from the energy equation. Using the downstream bed as the elevation datum gives:
Y = depth, m (ft)
V = velocity, m/s (ft/s)
Z = bed elevation referenced to a common datum, m (ft)
g = acceleration due to gravity 9.81 m/s^2 (32.2 ft/s^2)
Design Discharge Q = 5,900 ft^3/s
Channel Width B = 105 ft
Upstream Water Depth Y_u = 10.6 ft
Tail Water Depth d_m, Y_d = 9.5 ft
Unit Discharge q = 56.2 $ft^3/s/ft$
Upstream Mean Velocity V_u = 5.3 ft/s
Downstream Mean Velocity V_d = 5.9 ft/s
Drop Height h = 4.6 ft

H ft t= + +
(
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)
) |
− + +
(
\ |
)
) |
10 6 5 3 =
2 322
4 6 9 5 5 9
2 322
0 56
2 2
. (.)
( ) .
. . .
( ) .
. **EQ 35**

Using EQ 33, the estimated depth of scour below the downstream bed level is:

In this case, the unsupported height of the structure is ($h + d_s$) or 12.2 ft. If, for structural reasons, this height is unacceptable, then either install riprap to limit scour depth or a series of check dams could be constructed. It should be noted that if a series of drops are required, adequate distance between each drop must be maintained.

Lateral Scour Downstream of Check Dams

Lateral scour of the banks of a stream downstream of check dams can cause the streamflow to divert around the check dam. If this occurs, a head cut may move upstream and endanger the railroad crossing. To prevent this the banks of the stream must be adequately protected using riprap or other revetments. Riprap should be sized and placed in a similar fashion as for spurs and guide banks. The designer is referred to HEC-11 for proper sizing, and placement of riprap on the banks. Revetments are discussed in Article 3.6.4.5.

d_s $K_u H_t$ **EQ 36**
$0.225 q^{0.54} = - d_m$

Figure 1-3-60. Design Example of Scour Downstream of a Drop Structure

$d_s = 7.6$ ft

d_s $1.32(5.6)^{0.225}(56.2)^{0.54} = - 9.5$

References for Check Dam Design

Brown, S.A. and Clyde, E.S., 1989. "Design of Riprap Revetment," Hydraulic Engineering Circular No.11, FHWA-IP-89-016. Prepared for the Federal Highway Administration, Washington, D.C.
Federal Highway Administration, 1983. "Hydraulic Design of Energy Dissipators for Culverts and Channels," Hydraulic Engineering Circular Number 14, U.S. Department of Transportation, Washington, D.C.
Lagasse, P.F., Zevenbergen, L.W., Schall, J.D., and Clopper, P.E., 2001. "Bridge Scour and Stream Instability Countermeasures – Experience, Selection, and Design Guidelines," Second Edition, Report FHWA NHI 01-003, Federal Highway Administration, Hydraulic Engineering Circular No. 23, U.S. Department of Transportation, Washington, D.C.
Pemberton, E.L. and Lara, J.M., 1984. "Computing Degradation and Local Scour," Technical Guidelines for Bureau of Reclamation, Engineering Research Center, Denver, CO, January.
Peterka, A.J., 1964. "Hydraulic Design of Stilling Basins and Energy Dissipators," Engineering Monograph No. 25, Bureau of Reclamation, Division of Research, Denver, CO.

3.6.4.8 Channel Cutoffs

Design Considerations

For some railroad encroachments, a change in the river channel alignment is advantageous. When a river crossing site is so constrained by non-hydraulic factors that consideration of alternative sites is not possible, the engineer must attempt to improve the local situation to meet specific needs. Also, the engineer may be forced to make channel improvements in order to maintain and protect existing embankment in or adjacent to the river.

Suppose a meandering river is to be crossed with the alignment, as shown in Figure 1-3-61(a). Assume that the alignment is fixed by constraints in the acquisition of the right-of-way. To create better flow alignment with the bridge, consideration is given to channel improvement as shown in Figure 1-3-61(b). Similarly, consideration for improvement to the channel would also be advisable for a hypothetical lateral encroachment of a roadway as depicted in Figure 1-3-61(c). In either case, the designer's questions are how to realign the channel, and what criteria to use to establish stable channel dimensions.

Prior to realigning a river channel the stability of the existing channel must be examined. A stream classification, recent and past aerial photographs and field surveys are generally necessary (see Article 3.4.5).

The realigned channel may be made straight without curves, or may include one or more curves. If curves are included, the radii of curvature, the number of bends, the limits of rechannelization (hence the length or slope of the channel) and the cross-sectional area are decisions which have to be made by the designer. Different rivers have different characteristics and historical background with regard to channel migration, discharge, stage, geometry and sediment transport. As indicated in the previous chapters, it is important for the designer to understand and appreciate river hydraulics and geomorphology when making decisions concerning channel relocation. It is difficult to state generalized criteria for channel relocation applicable to every river. Knowledge about river systems has not yet advanced to such a state as to make this possible. Nevertheless, it is possible to provide some principles and guidelines for the design engineer.

As the general rule, the radii of bends (R_c) as shown in Figure 1-3-61(a) should be made about equal to the mean radii of bends in extended reaches of the river. When the angle ϕ defined in Figure 1-3-61(a) exceeds about 40 degrees, this provides a sufficient crossing length for the thalweg to shift from one side of the channel to the other.

Generally, it is necessary to stabilize the outside banks of the curves in order to hold the new alignment, and depending upon crossing length some amount of maintenance may be necessary to remove sandbars after large

floods so that the channel does not develop new meander patterns in the crossings during normal flows. Any designed increase in width should be limited to about 10 to 15 percent. Wider channels would be ineffective. Deposition would occur along one bank and the effort of extra excavation would be wasted. Furthermore, bar formation would be encouraged, with resultant tendencies for changes in the meander pattern leading to greater maintenance costs for bank stabilization and removal of the bars to hold the desired
river alignment. The depth of flow in the channel is dependent on discharge, effective channel width, sediment
transport rate (because it affects bed form and channel roughness) and channel slope.
This discussion pertains to alluvial channels with silt and sand sized bed materials. For streams with gravel and
cobble beds, the usual concern is to provide adequate channel cross-sectional dimensions to convey flood flows. If
the realigned channels are made too steep, there is an increase in transport rate of the bed material. The deposition of material in the reaches downstream of the crossing tends to form gravel bars and encourages changes
in the planform of the channel. Short-term changes in channel slope can be expected until equilibrium is reestablished over extended reaches both upstream and downstream of the rechannelized reach. Bank stabilization
may be necessary to prevent lateral migration, and periodic removal of gravel bars may also be necessary.

Assessment of Stability for Relocated Streams

Brice (1980) reported case histories for channel stability of relocated streams in different regions of the United
States. Based on his study, the recommendations and conclusions presented here apply to specific aspects of the planning and construction of channel relocation. They are intended for assessment of the risk of instability
and for reduction of the degree of instability connected with relocation. Serious instability resulting from relocation can be observed either when the prior natural channel is unstable or when floods of high recurrence

Figure 1-3-61. Encroachment on a Meandering River

interval occur during or soon after construction. Although there is an element of uncertainty in channel stability, the experience represented by Brice's study sites provides useful guidelines for improvement in the performance of relocated channels.

Channel Stability Prior To Relocation. Assessment of the stability of a channel prior to relocation is needed to assess erosion-control measures and risk of instability. An unstable channel is likely to respond unfavorably to relocation. Bank stability is assessed by field study and the stereoscopic examination of aerial photographs. The most useful indicators of bank instability are cut or slumped banks, fallen trees along the bankline, and wide, unvegetated, exposed point bars. Bank recession rates are measured by comparison of time-sequential aerial photographs. Vertical instability is equally important but more difficult to determine. It is indicated by changes in channel elevation at bridges and gaging stations. Serious degradation is usually accompanied by generally cut or slumped banks along a channel.

Erosion Resistance Of Channel Boundary Materials. The stability of a channel, whether natural or relocated, is partly determined by the erosion resistance of materials that form the wetted perimeter of the channel. Resistant bedrock outcrops, which extend out into the channel bottom, or that lie at shallow depths, will provide protection against degradation. Not all bedrock is resistant. Erosion of shale, or of other sedimentary rock types interbedded with shale, has been observed. Degradation was slight or undetected at most sites where bed sediment was of cobble and boulder size. However, serious degradation may result from relocation. Degradation may result from the relocation of any alluvial channel, whatever the size of bed material, but the incidence of serious degradation of relocated channels is slight.

The cohesion and erosion resistance of banks tend to increase with clay content. Banks of weakly coherent sand or silt are clearly subject to rapid erosion, unless protected with vegetation. No consistent relation was found between channel stability and the cohesion of bank materials, probably because of the effects of vegetation.
Length Of Relocation. The length of relocation contributes significantly to channel instability at sites where its value exceeded 250 channel widths. When the value is below 100 channel widths, the effects of length of relocation are dominated by other factors. The probability of local bank erosion at some point along a channel increases with the length of the channel. The importance of vegetation, both in appearance and in erosion control, would seem to justify a serious and possibly sustained effort to establish it as soon as possible on the graded banks.
Bank Revetment. Revetment makes a critical contribution to stability at many sites where it is placed at bends and along embankments. Rock riprap is by far the most commonly used and effective revetment (see Article 3.6.4.5).
Check Dams (drop structures). In general, check dams are effective in preventing channel degradation (see Article 3.6.4.7). The potential for erosion at a check dam depends on its design and construction, its height and the use of revetment on adjoining banks. A series of low check dams, less than about 2 ft in height, is probably preferable to a single higher structure, because the potential for erosion and failure is reduced. One critical problem arising with check dams relates to improper design for large flows. Higher flows have worked around the ends of many installations to produce failure.
Maintenance. The following problems that can be controlled by maintenance were observed along relocated channels: (1) growth of annual vegetation in channel; (2) reduction of channel conveyance by overhanging trees; (3) local bank cutting; and (4) bank slumping. The expense of routine maintenance or inspection of relocated channels beyond the right-of-way is probably prohibitive. However, most of the serious problems could be detected by periodic inspection, perhaps by aerial photography, during the first 5 or 10 years after construction.
Relationship Between Sinuosity And Stability. This relationship is summarized as follows: (1) Meandering does not necessarily indicate instability; an unstable stream will not remain highly sinuous for very long, because the sinuosity will be reduced by frequent meander cutoffs; (2) Where instability is present along a reach, it occurs mainly at bends; straight segments may remain stable for decades; and (3) The highest instability is for reaches whose sinuosity is in the range of 1.2 to 2 and whose type is either wide bend or braided point bar.

River Response to Cutoffs

The following three conceptual examples provide a summary of river response to cutoffs. In Table 1-3-19, each
individual case is identified in the first column to show the physical situation that exists prior to the cutoff. In the following three columns some of the major effects (local, upstream, and downstream) resulting from the cutoff at a particular crossing are given.

Case (1) illustrates a situation where artificial cutoffs have straightened the channel downstream of a particular crossing. Straightening the channel downstream of the crossing significantly increases the channel slope. This causes higher velocities, increased bed material transport, degradation and possible head cutting in
the vicinity of the structure. This can result in unstable river banks and a braided streamform. The straightening of the main channel can drop the base level, adversely affecting tributary streams flowing into the straightened reach of the main channel.

Case (2) illustrates a situation where the main channel is realigned in the vicinity of the bridge crossing. A cutoff is made to straighten the main channel through the selected bridge site. As discussed in Case (1), increased local gradient, local velocities, local bed material transport, and possible changes in the characteristics of the channel are expected due to the new conditions. As a result the channel may braid. A short cut off section (1 or 2 bends) can be designed to transport the same sediment loads that the river is capable of carrying upstream and downstream of the straightened reach; however, it may be difficult to achieve
stability when multiple bends in a long reach are cut off.

Table 1-3-19. River Response to Cutoffs (HDS 6)

Bridge Location Local Effects Upstream Effects Downstream Effects
(1) Cutoffs downstream of crossing
1 - Steeper slope
2 - Higher velocity
3 - Increased transport
4 - Degradation and
possible headcutting
5 - Banks unstable
6 - River may braid
7- Danger to bridge
foundation from
degradation and local
scour
See local effects 1 - Deposition downstream
of straightened
channel
2 - Increase in flood
stage
3 - Loss of channel
capacity
4 - Degradation in
tributary
(2) River channel relocation at crossing
site
1 - None if straight
section is designed to
transport the sediment
load of the river and if it is
designed to be stable
when subjected to
anticipated flow.
Otherwise same as in
Case (1) above
1 - Similar to local
effects
1 - Similar to local
effects
(3) Longitudinal encroachment
1 - Increased energy
gradient and potential
bank and bed scour
2 - Highway fill is subject
to scour as channel tends
to shift to old alignment
3 - Reach is subject to
bed degradation as
headcut develops at the
downstream end and
travels upstream
4 - Lateral drainage into
the river is interrupted and
may cause flooding and
erosion
1 - Energy gradient
also increased in
the reach upstream
and may cause
change of river form
from meandering to

braided
2 - Rate of sediment
transport is increased.
As the headcut travels
upstream severe bank
and bed erosion is
possible
3 - If tributaries in the
zone of influence exist
they will respond to
lowering of base level
1 - Channel will
aggrade as the
sediment load coming
from bed and bank
erosion is received
2 - Channel may
deteriorate from
meandering to braided

It is possible to build modified reaches of main channels that do not introduce major adverse responses due to local steepening of the main channel. In order to design a straightened channel so that it behaves essentially as the natural channel in terms of velocities and magnitude of bed material transport, it is usually necessary to build a wider, shallower section.

Case (3) illustrates an example of longitudinal encroachment. Here, a few bends of a meandering stream have been realigned to accommodate a railroad (see Case 2). There are two problems involved in channel realignment. First, the length of realigned channel is generally shorter than the original channel which results in a steeper energy gradient in the reach (Case 1). Second, the new channel bank material in the realigned reaches may have a smaller resistance to erosion. As a result of these two problems, the channel may suffer instability by the formation of a headcut from the downstream end and increased bank erosion. The realigned channel may also exhibit a tendency to regain the lost sinuosity and may approach and scour the railroad embankment. To counter these local effects one could design the realignment to maintain the original channel characteristics (length, sinuosity). Another way would be to control the slope by a series of low check dams. In any case, bank protection by riprap, jacks or spurs will be needed.

References for Channel Cutoffs

Brice, J.C., 1980. "Stability of Relocated Stream Channels," FHWA-RD-80-158, Federal Highway Administration, Washington, D.C.

Richardson, E.V., Simons, D.B., and Lagasse, P.F., 2001. "River Engineering for Highway Encroachments – Highways in the River Environment," Report FHWA NHI 01-004, Federal Highway Administration, Hydraulic Design Series No. 6, Washington, D.C.

3.6.4.9 Other Countermeasures

Introduction

Article 3.6.4.1 through Article 3.6.4.8 contain specific design procedures for a variety of stream instability and bridge scour countermeasures that have been applied successfully on a state or regional basis. Other countermeasures such as retarder structures, longitudinal dikes, bulkheads, and even channel relocations may be used to mitigate scour at bridges or stream bank erosion. Some of these measures are discussed and general guidance is summarized in this section.

Hardpoints

Hardpoints consist of stone fills spaced along an eroding bank line, protruding only short distances into the channel. A root section extends landward to preclude flanking. The crown elevation of hardpoints used by the USACE at

demonstration sites on the Missouri River was generally at the normal water surface elevation at the toe, sloping up at
a rate of about 1 ft in 10 ft toward the bank. Hardpoints are most effective along straight or relatively flat convex banks where the streamlines are parallel to the bank lines and velocities are not greater than 10 ft/s within 50 ft of the
bank line. Hardpoints may be appropriate for use in long, straight reaches where bank erosion occurs mainly from a
wandering thalweg at lower flow rates. They would not be effective in halting or reversing bank erosion in a meander
bend unless they were closely spaced, in which case spurs, retarder structures, or bank revetment would probably cost
less. Figure 1-3-62 is a perspective of a hardpoint installation. Hardpoints have been used effectively as the first "spur" in a spur field (see Article 3.6.4.4).

Retarder Structures

Retarder structures are permeable or impermeable devices generally placed parallel to streambanks to reduce velocities and cause deposition near the bank. They are best suited for protecting low banks or the lower portions of
streambanks. Retarder structures can be used to protect an existing bank line or to establish a different flow path or
alignment. Retards do not require grading of the streambank, and they create an environment which is favorable to the establishment of vegetation.

Jacks. Jacks most commonly consist of three linear members fixed together at their midpoints so that each member is perpendicular to the other two. Wires are strung on the members to resist distortion and to collect debris. Cables are used to tie individual jacks together and for anchoring key units to deadmen (Figure 1-3-63).

Jacks are effective in protecting banks from erosion only if light debris collects on the structures thereby enhancing their performance in retarding flow. However, heavy debris and ice can damage the structures severely. They are most effective on mild bends and in wide, shallow streams which carry a large sediment load.

Where jacks are used to stabilize meandering streams, both lateral and longitudinal rows are often installed to
form an area retarder structure rather than a linear structure. Lateral rows of jacks are usually oriented in a downstream direction from 45° to 70°. Spacing of the lateral rows of jacks may be 50 to 200 ft depending on the
debris and sediment load carried by the stream. A typical jack unit is shown in Figure 3.60 and a typical area installation is shown in Figure 1-3-64. Photographs of jacks and other arrangements that provide similar modus operandi can be found in a paper by Byers.

Outflanking of jack installations is a common problem. Adequate transitions should be provided between the upstream bank and the structure, and the jack field should be extended to the overbank area to retard flow velocities and provide additional anchorage. Jacks are not recommended for use in corrosive environments or at locations where they would constitute a hazard to recreational use of the stream.

Figure 1-3-62. Perspective View of Hardpoint Installation With Section Detail (after Brown)

Fence Retarder Structures. Fence retarder structures provide protection to the lower portions of banks of relatively
small streams. Posts may be of wood, steel, or concrete and fencing may be composed of wood planks or wire.

Figure 1-3-63. Typical Jack Unit (after Brown)

Figure 1-3-64. Retarder Field Schematic (after HDS 6)

Scour and the development of flow channels behind linear structures are common causes of failure of longitudinal fences. Scour at the supporting members of the structure can be reduced by placing rock along the fence or the effects of scour can be overcome by driving supporting members to depths below expected scour. Tiebacks can be used to retard velocities between the linear structure and the streambank, thus reducing the ability of the stream to develop flow channels behind the structure.

Timber Pile. Timber pile retarder structures may be of a single, double, or triple row of piles with the outside of the upstream row faced with wire mesh or other fencing material. They have been found to be effective at sharp bends in the channel and where flows are directly attacking a bank. They are effective in streams which carry heavy debris and ice loads and where barges or other shipping vessels could damage other countermeasures or a bridge. As with other retarder structures, protection against scour failure is essential. Figure 1-3-65 illustrates a design.

Wood Fence. Wood fence retarder structures have been found to provide a more positive action in maintaining an existing flow alignment and to be more effective in preventing lateral erosion at sharp bends than other retarder structures. Figure 1-3-66 is an end view of a typical wood fence design with rock provided to protect against scour.

Wire Fence. Wire fence retarder structures may be of linear or area configuration, and linear configurations may be of single or multiple fence rows. Double-row fence retards are sometimes filled with brush to increase the flow retardance. Figure 1-3-67 and Figure 1-3-68 illustrate two types of wire fence retarder structures.

Longitudinal Dikes

Longitudinal dikes are essentially impermeable linear structures constructed parallel with the streambank or along the desired flow path. They protect the streambank in a bend by moving the flow current away from the bank. Longitudinal dikes may be classified as earth or rock embankment dikes, crib dikes, or rock toe-dikes.

Figure 1-3-65. Timber Pile Bent Retarder Structure (after Brown)

Figure 1-3-66. Typical Wood Fence Retarder Structure (after Brown)

Figure 1-3-67. Light Double Row Wire Fence Retarder Structure (after Brown)

Earth or Rock Embankments. As the name implies, these dikes are constructed of earth with rock revetment or of rock. They are usually as high or higher than the original bank. Because of their size and cost, they are useful only for large-scale channel realignment projects.

Rock Toe-Dikes. Rock toe-dikes are low structures of rock riprap placed along the toe of a channel bank. They are useful where erosion of the toe of the channel bank is the primary cause of the loss of bank material. The USACE has found that longitudinal stone dikes provide the most successful bank stabilization measure studied for channels which are actively degrading and for those having very dynamic beds. Where protection of higher portions of the channel bank is necessary, rock toe-dikes have been used in combination with other measures such as vegetative cover and retarder structures.

Figure 1-3-68. Heavy Timber-pile and Wire Fence Retarder Structure (after Brown)

Figure 1-3-69 shows the typical placement and sections of rock toe-dikes. The volume of material required is 1.5 to 2 times the volume of material that would be required to armor the sides of the anticipated scour to a thickness of 1.5 times the diameter of the largest stone specified. Rock sizes should be similar to those specified
for riprap revetments (see Article 3.6.4.5). Tiebacks are often used with rock toe-dikes to prevent flanking, as illustrated in Figure 1-3-70. Tiebacks should be used if the toe-dike is not constructed at the toe of the channel
bank.

Rock toe-dikes are useful on channels where it is necessary to maintain as wide a conveyance channel as possible. Where this is not important, spurs could be more economical since scour is a problem only at the end
projected into the channel. However, spurs may not be a viable alternative in actively degrading streams (see Article 3.6.4.4).

Crib Dikes. Longitudinal crib dikes consist of a linear crib structure filled with rock, straw, brush, automobile tires or other materials. They are usually used to protect low banks or the lower portions of high banks. At sharp bends, high banks would need additional protection against erosion and outflanking of the crib dike. Tiebacks can be used to counter outflanking.

Crib dikes are susceptible to undermining, causing loss of material inside the crib, thereby reducing the effectiveness of the dike in retarding flow. Figure 1-3-71 illustrates a crib dike with tiebacks and a rock toe on the stream side to prevent undermining.

Bulkheads. Bulkheads are used for purposes of supporting the channel bank and protecting it from erosion. They are generally used as protection for the lower bank and toe, often in combination with other countermeasures that provide protection for higher portions of the bank. Bulkheads are most frequently used at bridge abutments as protection against slumping and undermining at locations where there is insufficient space for the use of other types of bank stabilization measures, and where saturated fill slopes or channel banks
cannot otherwise be stabilized.

Bulkheads are classified on the basis of construction methods and materials. They may be constructed of concrete, masonry, cribs, sheet metal, piling, reinforced earth, used tires, gabions, or other materials. They must be protected against scour or supported at elevations below anticipated total scour, and where sections of
the installation are intermittently above water, provisions must be made for seepage through the wall. Some bulkhead types, such as crib walls and gabions, should be provided with safeguards against leaching of materials from behind the wall.

Bulkheads must be designed to resist the forces of overturning, bending and sliding, either by their mass or by
structural design. Figure 1-3-72 illustrates anchorage schemes for a sheetpile bulkhead. Because of costs, they should be used as countermeasures against meander migration only where space is not available to construct other types of measures.

Figure 1-3-69. Typical Longitudinal Rock Toe-dike Geometries (after Brown)

Figure 1-3-70. Longitudinal Rock Toe-dike Tiebacks (after Brown)

Figure 1-3-71. Timber Pile, Wire Mesh Crib Dike With Tiebacks (after Brown)

Figure 1-3-72. Anchorage Schemes for a Sheetpile Bulkhead (after Brown)

References for Other Countermeasures

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Richardson, E.V., Simons, D.B., and Lagasse, P.F., 2001. "River Engineering for Highway Encroachments – Highways in the River Environment," Report FHWA NHI 01-004, Federal Highway Administration, Hydraulic Design Series No. 6, Washington, D.C.

SECTION 3.7 MEANS OF PROTECTING ROADBED AND BRIDGES FROM WASHOUTS AND FLOODS

3.7.1 GENERAL (1996)

Adequate protection against floods and washouts is essential not only for maintenance of dependable service, but also to avoid heavy expenditures to replace damaged facilities and restore operation.

3.7.2 ROADWAY (1996)

3.7.2.1 General – Risks and Possible Damage

Water overflowing the embankment, either from a direct flow or backwater, frequently results in damage to the roadway. This damage may be as severe as a washout or less apparent in other forms, such as, a loss of the shoulder, a steepening of the embankment, a loss of crib or shoulder ballast, or a softening of the subgrade's support characteristics.

Damage resulting from sloughing and slides are usually more severe as the water recedes from a saturated embankment. Loose, fine-grained, cohesionless soils are more susceptible to sloughing. In general, soil conditions, vegetation, and the rapidity at which the water recedes are primary factors in determining the risk of sloughing.

3.7.2.2 Temporary Protection Measures

a. Temporary protection of the roadway section is sometimes necessary, particularly in flood events where immediate action is necessary and time constraints do not permit implementation of a permanent solution. Periodic and close track inspections of flood and washout susceptible areas and identification of high risk locations will be a beneficial first step in determining the appropriate remedial repair.

b. Temporary protection of potential overflow slopes and fill sections subject to erosion and sloughing can be provided by placement of an armor of heavy weight material, not easily displaced by floodwaters, such as large-sized stone (riprap) or sandbags. In blanketing the slopes, it is critical that the toe be adequately protected to minimize the risk of base scour and possible embankment failure. Raising the roadbed shoulder with riprap and sandbags can also be a suitable means for temporary relief.

3.7.2.3 Permanent Protection Measures

a. In overflow territories, care must be taken to review the adequacy of design, location and construction of existing drainageways and make appropriate corrections if deficiencies are found. Sufficient waterway

capacity is essential to minimize heading during floods and, if necessary, provisions should be made for additional relief openings to handle the flow. The impact of runoff from neighboring facilities, existing and proposed, must also be assessed. Input from applicable local, state, or federal authorities should be sought in these preliminary drainage assessments.

b. Selection of the optimal permanent protection measure should be done on a site-specific basis and will depend on many factors, including service requirements, severity and extent of the damage potential, embankment soil characteristics, and economic considerations. A subsurface exploration of the area in question, performed during the preliminary stages, can many times generate valuable information and aid in the selection and design process.

c. In general, depending upon service requirements, a track raise is the best assurance for reliable operation. Roadbeds subject to severe side erosion can be protected by relocation of the track and/or channel, or construction of revetments as discussed in Article 3.4.5 and Article 3.4.7, respectively. In overflow bottoms where either a channel change, installation of additional openings, or a track raise or relocation do not afford sufficient relief, consideration should be given to facing the downstream side of the roadbed at least at critical locations with riprap or other suitable means of protection. Covering erosion-susceptible slopes with a thick vegetative cover can furthermore provide protection by impeding surface erosion.

d. On light traffic density lines where the aforementioned extensive measures cannot be economically justified, consideration might be given to anchoring the track to the roadbed, at designated locations throughout the overflow area, utilizing cable tied to rail, timber pile, or screw anchors driven in the roadbed. Under these conditions use of a heavy course ballast tends to reduce the incidence of ballast displacement. When using this last method of protection, the railroad is accepting the risk of traffic disruption due to flooding and washouts.

3.7.3 BRIDGES (1996)

3.7.3.1 General – Risks and Possible Damage

Protection against flood damage for structures calls for resourcefulness during the immediate flooding threat, as well as during the implementation of permanent protection measures. Temporary measures should be given consideration to prevent both minor and major damage. Minor damage can be categorized as scour on the shoulders or behind the abutments, debris hung up in the waterway opening, overtopping, and various other damage that can be immediately detected and repaired. Major damage are items such as contamination of ballast decks and roadbed; scouring around piling, piers, foundations, and backwalls; channel changes resulting in silting or bypassing the structure; culvert piping or joint separation; etc.

3.7.3.2 Temporary Protection Measures

The need for temporary protection should be considered not only prior to and during floods, but also when the structure is under construction. Temporary measures to consider during or immediately preceding a flood include, identification of high-risk areas, frequent inspection, remove or pass debris through the structure to avoid accumulation, and the placement of riprap or sandbags. The following are temporary measures to be considered when the structure is in the design or construction stage; all the measures considered previously, and others such as fence jetties, rock jetties, and channel cutoffs.

3.7.3.3 Permanent Protection Measures

Permanent protection measures require that sound engineering principles be employed to protect the structure from flood damage and allow its continued function as designed. Bridges and culverts must be designed with sufficient waterway opening to handle the design storm. In addition, both structures must be designed with an adequate opening to pass the anticipated debris. This also requires occasional re-evaluation as the drainage area or other conditions change. Some of the measures detailed in various articles in Section 3.4, Basic Concepts and Definitons of Scour, may need to be incorporated in the structures protection plan. Permanent protection might also include underwater or other inspections of potential problem areas.

SECTION 3.8 CONSTRUCTION AND PROTECTION OF ROADBED ACROSS RESERVOIR AREAS[1]

3.8.1 GENERAL (1978)

a. The construction and protection of roadbed across reservoir areas present many problems that are not encountered in normal roadbed construction. Analysis of these problems can best be made by subdividing the subject into three sections, as follows:

• Determination of Wave Heights

• Construction of Embankment and Roadbed
• Construction of Embankment Protection
b. The term “reservoir area” as used in this report also includes lakes, natural and artificial river pools, and other inland waters on which waves may be generated.

3.8.2 DETERMINATION OF WAVE HEIGHTS (1978)

a. Knowledge relating to wind velocities over land and over water, and wave heights on inland reservoirs, has increased in recent years as a result of studies made by the Coastal Research Center (formerly known as the Beach Erosion Board), and by the Corps of Engineers at Fort Peck Reservoir in northeast Montana, Denison Reservoir on the Oklahoma-Texas state line, and Lake Okeechobee in Southern Florida.
b. These studies resulted in the publication of Technical Memorandum No. 132, “Waves in Inland Reservoirs” (Reference 4) by the Beach Erosion Board, essentially the same information having appeared in ASCE Proceedings Paper No. 3138 (May 1962), corrected May 1963.
c. The methods subsequently described are adapted from Technical Memorandum No. 132, and are adequate for ordinary wave problems. For more extensive or complicated situations, the designer should refer to Technical Memorandum No. 132, or to Technical Report No. 4, “Shore Protection Planning and Design” (Reference 5) by the Beach Erosion Board, or to other references listed in these publications.
d. Elements affecting the determination of wave heights may be listed as follows.

3.8.2.1 Effective Fetch (F)

a. Fetch, or the distance over which the wind blows, was originally designated as the greatest straight-line distance across open water. Subsequent studies have shown that the shape of an open-water area affects the effective fetch.
b. For a given size and shape of water area, effective fetch is determined by laying out seven radials at 6 degrees intervals on each side of a central line through the point under study, extending them to their point of intersection with the shore line. The scaled component of each radial’s projection on the central radial is then multiplied by the cosine of its angle with the central radial. The sum of these values divided by the sum of the cosines determines the effective fetch (F) for that location. An example of this calculation is shown in Figure 1-3-75.

[1] References, Vol. 56, 1955, pp. 706, 1118; Vol. 57, 1956, pp. 649, 1080; Vol. 63, 1962, pp. 578, 749; Vol. 66, 1965, pp. 523, 746; Vol. 78, 1977,
p. 124.

3.8.2.2 Wind Velocity (U)

a. Wind velocities over water are higher than over land, and although individual observed values may vary considerably, average values for this relationship have been observed as shown in Table 1-3-20.
b. Thus, a wind having a velocity of 40 mph over land could be expected to attain a velocity of 40 × 1.28, or 51 mph over water if the effective fetch were 4 miles.

3.8.2.3 Minimum Wind Duration (t_a)

With wind velocity assumed to be constant over a particular fetch, the height of waves being generated will progressively increase with time up to a maximum value for that velocity. The minimum wind duration in minutes for producing this maximum wave can be determined from the dashed lines in Figure 1-3-73, given the wind velocity in miles per hour and the effective fetch in miles.

3.8.2.4 Significant Wave Height (H_s)

Although successive waves in a group will vary in height, the significant wave height is defined as the average of the highest one-third of the waves being generated, measured from trough to crest, and is determined from the solid diagonal lines in Figure 1-3-73. Since wind-generated waves on a large body of water are not uniform in height, the significant wave height thus determined will be exceeded approximately 13% of the time.

3.8.2.5 Specific, or Design, Wave Height (H_o)

a. Wave studies have shown that wind-generated waves are not uniform in height, but consist of groups of waves with varying heights. Studies of inland reservoirs show the following relationship between the significant wave height (H_s) which is exceeded 13% of the time, and a selected specific wave height (H_o) exceeded less frequently (Table 1-3-21).
b. Having determined the significant wave height from Figure 1-3-73, a design wave of acceptable frequency of occurrence is computed by multiplying H_s by the corresponding ratio value in Table 1-3-21.

A ratio of 1.87 is frequently used for the so-called maximum wave, but over extended periods of observation, individual waves may even exceed this value.

Table 1-3-20. Wind Relationship – Land to Water

Fetch in Miles	Wind ratio
0.5	1.08
1	1.13
2	1.21
4	1.28
6 and over	1.31

Uwater
Uland

Table 1-3-21. Wave Height Distributions

Ratio of Specific Wave Height H_o to Significant Wave Height H_s (H_o /H_s) (1)	Percent of Waves Exceeding Specific Wave Height H_o (2)
1.00	13
1.07	10
1.27	4
1.40	2
1.60	1
1.67	0.4

Figure 1-3-73. Wave Heights and Minimum Wind Durations

3.8.2.6 Wave Period (T)

The significant wave period represents the average interval in seconds between successive waves, and is determined from Figure 1-3-74. The resulting wave period is also applicable to the higher waves in the group.

3.8.2.7 Wave Length (L)

a. The wave length (L) is measured from crest to crest of waves in feet, and is equal to 5.12 T^2. However, wave heights and other characteristics are limited by the depth of water in which they are generated if that depth is less than approximately one-half the wave length. Observations at Lake Okeechobee (Reference 32) indicated that waves were limited to a maximum significant wave height of approximately 0.6 of the average depth of water in the generating area, regardless of duration and velocity of wind.

b. For determining the characteristics of shallow-water-generated waves, reference may be made to the previously mentioned Technical Memorandum No. 132, or Technical Report No. 4.

Figure 1-3-74. Wave Periods

3.8.2.8 Set-up, or Wind Tide (S)

a. An enclosed body of water over which a wind is blowing tends to pile up at higher elevations at the leeward end, and on long fetches this set-up may assume importance in the design of bank protection. The increase in water level above the still-water elevation that would prevail without wind action is determined by the formula:

where:

b. Fetch distance used here differs from effective fetch (F) previously described in that it may be of a curved or sweeping character, and need not move in a straight line. It can also be greater in length than the effective fetch (see Figure 1-3-75).

S = set-up in feet

U = velocity of the wind in miles per hour

F = fetch in statute miles

D = average depth of the body of water in feet

S U^2F

1400D

= -----------------

Figure 1-3-75. Fetch Example Calculation

3.8.2.9 Wave Run-up (R)

On readying a fill a wave will run up the slope to an elevation largely dependent on the angle of the slope, the roughness of the embankment, and the steepness of tide wave (H_o/L_o). From Figure 1-3-76, relative run-up on

smooth or average riprap covered slopes can be determined where the embankment slope and steepness of design wave are known. Run-up (R) in feet is secured by multiplying the relative run-up thus found by the design wave height (H_o), and total run-up will then be the sum of run-up (R) and set-up (S).

Figure 1-3-76. Wave Run-up Ratios

3.8.2.10 Summary

For a sample computation for determining wave height and protection needs see Table 1-3-22 and Figure 1-3-
75.

3.8.3 CONSTRUCTION OF EMBANKMENT AND ROADBED (1978)

a. The embankment should be constructed in accordance with Part 1, Roadbed, except as modified in conformance with the following:

b. That portion of the embankment which will be submerged should have side slopes not steeper than 3 to 1, and no material should be used in the embankment which has a liquid limit in excess of 60 as determined in accordance with ASTM Designation D 423-61T, Standard Method of Test for Liquid Limit of Soils. Width of roadbed, side slopes, prepared ballast and sub-ballast should be in accordance with the standards of the railroad company. Factors not encountered in ordinary roadbed construction that should receive consideration include probable maximum water-surface elevation, frequency of occurrence and duration of embankment submergence, possible head on the embankment (water surface on one side higher in elevation), and drawdown effect due to the rapid release of stored water.

Table 1-3-22. Sample Computation for Determining Wave Height and Protection Needs (See Figure 1-3-75)

Description Amount

1. Effective fetch (F) 1.42 miles

2. Average wind velocity over land 40 mph

3. Average wind velocity over water (U), = 40 × 1.21 from Table 1-3-20 48 mph
4. Minimum duration (t_d) to produce computed wave, from Figure 1-3-73 23 min
5. Significant wave height (H_s), from Figure 1-3-73 2.6 ft
6. Design wave height (H_o), exceeded by only 0.4% of the waves, = H_s + 1.67 from Table 1-3-21 4.3 ft
7. Wave period (T) for significant and design waves, from Figure 1-3-74 3.0 sec
8. Wave length (L_o) for design wave = 5.12 × T^2 46.1 ft
9. Wave steepness (H_o/L_o) = 4.3/46.1 0.093
10. Set-up (S) = , where wind tide fetch (F_s) is 6.24 miles (Figure 1-3-75), and average
depth of lake is 30 ft
0.34 ft
11. Relative run-up ratio (R/H_o) for riprap on 2.5:1 slope, and H_oL_o = 0.093, using Figure 1-3-76 0.94
12. Wave run-up, (R), (line 6 × line 11) 4.0
13. Total run-up, (line 10 + line 12) 4.3 ft
Weight of rock protection, from Article 3.8.4.2,
where:
S = 2.6 for limestone
H = 4.3, design wave height
cot a = 2.5 for 2.5:1 slope
then:
W_{avg}
W_{max}
W_{min}
Minimum Thickness
162 lb
648 lb
20 lb
18 in
U^2F_s
1400 D
----

c. The probable maximum water surface elevation, together with the design wave height, wind tide and wave run-up effects will govern the elevation at which the roadbed should be constructed. In special cases the increased headwater elevation due to inflowing water may also be significant.
d. Frequency and duration of embankment submergence is important in determining the suitability of available embankment material and recommended slopes. An embankment that is stable under ordinary conditions may not be stable when saturated because of long submergence.
e. The permeability of the soil to be used in the embankment should be considered in connection with the probable maximum rate of drawdown in the case of reservoirs. Embankments composed of impervious materials that are stable when dry, or even when saturated, may fail if the water surface is lowered rapidly while the embankment is saturated. Pervious free-draining soils are not as susceptible to failure from this cause as are impervious soils.

3.8.4 CONSTRUCTION OF EMBANKMENT PROTECTION (1978)

3.8.4.1 General

Experience has shown that in the majority of cases, dumped riprap furnishes the best type of protection at the lowest ultimate cost. Its effectiveness depends on the quality of the rock and its weight or size, thickness of the
layer, shape of the individual stones, slope of the embankment, and stability of tile base or filter on which it is placed. Usually, the availability of riprap sources determines to some extent the quality and size of stone used for slope protection.

3.8.4.2 Weight and Thickness of Riprap

a. Formulas for rock protection of slopes have, until recent years, become more applicable to costal

installations. Requirements for inland bodies of water have been found to require somewhat different standards of design. One extremely useful set of formulas appears in the Corps of Engineers' Manual, EM 1110-2-2300 (1 April 1959), and is substantially as follows:
Minimum Thickness in Inches =
where:
b. Gradation of weights of stone should fall within the following classifications: At least 45% shall be greater than W_{avg} with not more than 10% greater than W_{max} or 10% less than W_{min}.
W = weight of individual stones in pounds
S = specific gravity of the rock
H_o = design wave height in feet
a = the angle of slope with the horizontal
W_{avg}
62.4 S H_o
2
1.82 $(S – 1)^3$ cot a
= ---
W_{max} = 4 ¥ W_{avg}
W_{max}
W_{avg}
8'
= --------------
18
W_{avg}
62.4 S
3 ------------------

c. It should be noted that riprap selected by the use of these formulas is suitable for protecting embankments against wave action in normal circumstances. Special consideration should be given to specific needs for protection of foundations from scour, or at locations where riprap may be displaced by ice action.
d. All stone for riprap shall be preferably of angular and irregular shape, and of a quality that will reasonably withstand the action of water, frost, or other weathering.

3.8.4.3 Minimum Requirements

a. The protective covering should extend from the natural ground surface at the toe of slope to an elevation at least 2.0 feet above the height of total run-up as determined in Article 3.8.2.9, or 4.0 feet above stillwater elevation, whichever is greater. Where the natural ground does not provide adequate support, or where scour is possible, riprap should be extended below the toe of slope by trenching to the required depth.
b. The thickness of riprap cover should satisfy design requirements of Article 3.8.4.2, but should not be less than 18 inches thick.

3.8.4.4 Filter Blanket

a. A bedding layer or filter blanket should be provided underneath riprap protection when the compacted material of the underlying embankment consists of silt or fine sand. In this case there is danger of the fill material being washed out through voids in the riprap by wave action which can result in undermining of the cover material.
b. The filter blanket, composed of gravel (preferably crushed), crushed rock or slag, should be not less than 6 inches and not more than 12 inches in thickness, and should be placed on the embankment slope to form a backing for the riprap protection, and should be reasonably well graded within the limits found in Table 1-3-23.

3.8.4.5 Littoral Currents and Refraction

Littoral current, the result of waves breaking at an angle to the shoreline, and refraction, the process by which the direction of a wave moving in shallow water at an angle to bottom contours is changed, are primarily the concern of designers of coastal shore-protection structures, and are not covered here. Reference is made to the
Beads Erosion Board's Technical Report No. 4, "Shore Protection Planning and Design," for detailed information on these subjects.

Table 1-3-23. Filter Blanket Limits

Sieve Size	Percent by Weight Passing
3 inch	100
1-1/2 inch	40-60
No. 40	0-5

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FEDERAL RAILROAD ADMINISTRATION (FRA) **1997-2008**
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Railroad Safety Oversight Manager (RSOM)

Facilitated and coordinated a full range of safety programs, policies, projects, and initiatives through both labor and management on CSX Transportation Company to resolve both regulatory and non-regulatory systemic safety issues that impacted the overall safety and effectiveness of the CSX Railroad through the SACP process.

- Served as a cross-discipline safety technical expert, planning and implementing and evaluating various programs in the Office of Safety.
- Addressed railroad management and labor issues and integrated new initiatives into the Office of Safety's enforcement and compliance activities.
- Worked with labor and management to resolve safety issues by leading and coordinating numerous teams in a full range of programs, policies, projects, and safety initiatives that impact the overall safety of the railroad, its employees, and the general public.
- Served as a multi-skilled management and safety generalist, facilitating the planning, directing, and evaluation of Federal Railroad Administration Office of Safety initiatives.
- Provided technical and administrative guidance to field personnel and represent the FRA Associate Administrator for Safety on field issues.
- Signal and Train Control Electronic Recordkeeping: Lead the development of a program that ensured that federally required signal tests and inspections are performed and recorded.
- Track Compliance Agreement: Instrumental in recognizing and recommending that CSXT come under the safety agreement to ensure that CSXT addressed track inspection practices, track maintenance and repair procedures, and the effectiveness of quality control procedures regarding these programs.
- CSXT/UTU/FRA Safety Model - Worked in conjunction with management and labor to develop a model safety program, which empowers labor to manage and be accountable for its own safety program and implemented the program system wide.

- National Hazardous Material Audit- Lead the national hazardous material audit to determine Class I's compliance with train documentation and standing order within the trains.
- CSXT Harassment and Intimidation Audit – Lead a system-wide audit to determine CSXT's compliance with CFR Part 225.33.
- Maintenance of Way Manpower Audit – Lead an audit on the CSXT Huntington Division to determine if staffing levels and resource allegations were sufficient.

FEDERAL RAILROAD ADMINISTRATION **1996-1997**
Chief Inspector/Principle Regional Inspector **Jacksonville, FL**
Performed broad, multifunctional tasks, encompassing enforcement, drafting, and interpretation of federal railroad safety regulations (49 CFR), and provided internal/external guidance on the scope, applicability, intent and effect of FRA regulations.

- Responsible for Short Line Railroad compliance with all applicable Federal regulations in Florida, Georgia, South Carolina, and Alabama.
- Conducted hi-rail and walking inspections of the CSXT and Short Line Railroads to determine railroad carriers compliance with CFR Part 213.
- Conducted complaint investigations concerning safety issues for rail labor and concerned citizens within FRA Region 3.
- Conducted investigations employee injuries/fatalities, train derailments, accidents, and collisions often resulting in serious injury to persons and/or railroad property.
- Served as an advisor to the Regional Administrator, and Track Specialist addressing a broad range of issues, including accident investigation, multi-regional railroad compliance, and special investigations.
- Provided training in Track Safety Standards, 49 CFR Part 213 and CFR Part 214 to railroad employees in FRA Region.
- Technical Expert at FRA Claims Collection Settlement Conferences with the Class I, Regional, and Short Line railroads regulated by the FRA.

FEDERAL RAILROAD ADMINISTRATION (FRA) **1995-1996**
Deputy Regional Administrator (DRA) **Sacramento, CA**

As the Deputy Regional Administrator for the Southeast Region (Region 7), served as the principal technical advisor to the Regional Administrator on all aspects of regional railroad safety, operational, and administrative programs, encompassing four western states.

- Functioned as a multi-skilled manager and safety generalist, participating in the planning, directing, and evaluating of all Region 7 programs.
- Supervised the implementation of the National and Regional enforcement policies and programs to determine carrier compliance with federal safety laws, rules, and regulations pertaining to railroad safety.
- Managed, directed, and served as the first-line supervisor for Safety Specialists in five separate disciplines; i.e., Motive Power & Equipment, Operating Practices, Track, Signal and Train Control, and Hazardous Materials, as well as Grade Crossing Safety Managers.
- Managed approximately 45 FRA Federal and State Safety Inspectors, administrative staff, and IT specialists.
- Work with NTSB on major accidents to determine accident cause.
- Managed the Region 7 budget and payroll, including oversight of operational and travel funding and allocation.
- Spearheaded the day-to-day Region 7 operations, overseeing regional railroad safety inspection, enforcement and compliance activities, accident investigations, and waiver and complaint investigations.
- Served as National Safety and Assurance Program manager over Amtrak and facilitated and coordinated a full range of programs, policies, projects, and initiatives through both labor and management to resolve systemic safety issues.
- Also provided investigative coverage in multiple Region 7 railroad accidents, incidents, and fatalities.
- Critique and edit FRA inspectors' accident and complaint reports for technical accuracy.
- Served on multiple railroad safety compliance team initiatives.

FEDERAL RAILROAD ADMINISTRATION **1989-1995**
District Chief/Chief Inspector **Denver, CO**
Performed broad, multifunctional tasks, encompassing enforcement, drafting, and interpretation of federal railroad safety regulations (49 CFR), and provided internal/external guidance on the scope, applicability, intent, and effect of FRA regulations.

- Managed 6 FRA Safety Inspectors and 2 State Safety Inspectors in the Midwest (Region 6).
- Organized and led the first Hazardous Material Task Force Audit involving FHWA Motor Carrier Safety, Colorado State Patrol, U.S. Customs, and Colorado State Police.
- Technical Expert in developing and serving for several industry-wide Track Technical Resolution Committees (TRC's).
- Served as the FRA Technical Resolution Committee Chairman and served as a liaison between management and Office of Safety personnel.
- Served as the Team Leader for the MP&E Quality Team to facilitate the resolution of safety and non-compliance issues between Brotherhood of Railway Carmen and the Southern Pacific Railroad.
- Served on the Safety Assurance and Compliance Program (SACP) development team which developed the definitions, flow chart, and scope of the SACP. Wrote job descriptions for each of the positions within the SACP projects.
- Consultant/Advisor to senior management in a variety of government and private sector organizations for the FRA.
- Technical Expert at FRA Claims Collection Settlement Conferences with the Class I, Regional, and Short Line railroads regulated by the FRA.
- Work with NTSB on major accidents to determine accident cause.
- Served as a liaison for the FRA in numerous states to affect highway/rail grade crossing closures, which included a $3 million project involving 16 grade crossings in Ft. Collins, Colorado.

FEDERAL RAILROAD ADMINISTRATION **1984-1989**
Track Safety Inspector **Milwaukee, WI/Chicago, IL**
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- Conducted hundreds of hi-rail and walking inspections yearly to determine railroad carriers compliance with CFR Part 213.
- Conducted complaint investigations concerning safety issues rail labor and concerned citizens.
- Conducted investigations on employee injuries/fatalities, train derailments, accidents, and collisions often resulting in serious injury to persons and/or railroad property.
- Served as an advisor to the Regional Administrator, District Chief and Track Specialist addressing a broad range of issues, including accident investigation, multi-regional railroad compliance, and special investigations.
- Work with NTSB on major accidents to determine accident cause.
- Provided training in Track Safety Standards, 49 CFR Part 213 to railroad employees in Illinois, Wisconsin, and Michigan.
- Served as the Operation Lifesaver Coordinator and initiated programs with McDonalds Corporation in 17 states, resulting in the adoption of the program by Operation Lifesaver Inc in 1992.
- Received the Secretary of Transportation Award for my efforts in highway/rail grade crossing safety.

Missouri Public Service Commission **1983-1984**
Track Safety Inspector **Jefferson City, Missouri**
Responsible for the improvement and advancement of railroad safety in areas related to design, construction, inspection, maintenance, and use of railroad rolling stock and related appurtenances.

- Conducted over one hundred hi-rail and walking inspections to determine railroad carriers' compliance with CFR Part 213.
- Conducted complaint investigations concerning safety issues for rail labor and concerned citizens.
- Conducted investigations on employee injuries/fatalities, train derailments, accidents and collisions often resulting in serious injury to persons and/or railroad property.
- Served as an advisor to the State of Missouri program director.
- Provided training in Track Safety Standards, 49 CFR Part 213 to railroad employees in Missouri.
- Served as a member of the Missouri Operation Lifesaver program.

Inland Steel Coal **1980-1983**
Track Supervisor **McLeansboro, IL**

- Supervised track gangs in the installation of railroad track in the coal mine.
- Responsible for maintenance and repair of existing trackage.
- Trained personnel in the proper use of track tools and equipment.
- Initiated the integration of railroad safety rules with existing mining policy and assisted the engineering staff on standards and procedures for underground track work.
- Applied my experience and knowledge of the railroad industry to the coal mining industry.

Southern Railway Company **1976-1980**
Track Supervisor **Mt. Vernon, Illinois**

- Responsible for an approximate $1,750,000 inventory budget of material and equipment.
- Supervised a minimum of 21 employees, including 16 laborers, three foremen, and two assistant track supervisors.
- Supervised these employees in their track maintenance renewals, and repairs of 100 miles of Class 4 main line trackage and one terminal yard to ensure compliance with Southern Railway design and maintenance standards and FRA Track Safety Standards.
- Administered safety programs and operating rules tests; purchased and approved invoices for materials.

Southern Railway Company **1974 - 1976**
Timber and Surface Supervisor **Princeton, IN**

- Managed a mechanized gang that installed ties and surfaced track out-of-face in a five-state area in accordance with Southern Railway maintenance and design standards and within the scope of the Federal Track Safety Standards.
- Responsible for purchases necessary to the operation of the gang and the camp cars, which housed the employees.
- Administered safety programs and operating rules tests.

Southern Railway Company **1973-1974**
Assistant Track Supervisor **Clinton, TN**

- Inspected track daily by motor car and walking inspection for FRA compliance with CFR Part 213 Track Safety Standards.
- Assisted the Track Supervisor in all track maintenance, safety programs, training, rules tests, and materials purchasing.

Southern Railway Company **1972 -1973**
Assistant Track Supervisor Trainee **Princeton, IN**

- Responsible for invoices, purchase orders, and special projects in the engineering office.
- Relieved the Assistant Track Supervisors and Track Supervisors in their absence and inspected track by motor car and walking inspection for FRA compliance with CFR Part 213 Track Safety Standards.
- Participated in three Roadway Engineering classes and Management Skills workshops as part of the training program.

Louisville and Nashville Railroad **1970-1972**
Southern Railway Company **Princeton/Evansville, IN**
Track Laborer

- Performed track maintenance and repairs in accordance with railroad design and maintenance plans and in compliance with CFR Part 213.

EDUCATION

- Masters of Business Administration, Jacksonville University, 2002, Jacksonville, Florida

CERTIFICATIONS

- Project Management Professional (PMP) - Project Management Institute -2006
- Certified Track Inspector, Federal Railroad Administration, Department of Transportation

PROFESSIONAL AFFILIATIONS

- Member, American Railway Engineering and Maintenance of Way Association, 1978-present

SPECIALIZED PROFESSIONAL TRAINING

- Railroad Track Inspection & Safety Standards – Center for Transportation Research, The University of Tennessee – 2018
- OSHA 6010 - Occupational Safety and Health Course for Other Federal Agencies - 2016
- Orgo Thermit Welding Training - 2014
- FRA Track Recurrency Training CFR Parts 213/214 – 2014
- PMI Risk Management Concepts and Guidance – 2012
- FRA Track Recurrency Training CFR Part 213 – 2011
- FEMA ICS - 100 Training - 2009
- FEMA ICS - 200 Training - 2009
- FEMA National Incident Management System - 2009
- DOT/FRA National Workshops on Operating practices, Hazardous materials, Alcohol and Drug, and Track -2007
- DOT/FRA Track Recurrency - 2007
- Project Management Exam Course – 2006
- Management Development Center: Strategic Leadership–Building Performance-Based Organizations - 2006
- OPM Developing High Performing Teams - 2006
- OPM Leading Successful Project Teams - 2006
- OPM Optimizing Project Performance - 2006
- Project Management Principles – 2006
- DOT/FRA Privacy 101 – Privacy laws and regulations - 2006
- DOT/FRA Operational Testing and Human Performance Science -2005
- DOT/FRA Computer Security Awareness Training – 2005
- Management Development Center: Developing High Performing Teams – 2005
- DOT/FRA Technical Writing Class – 2003
- DOT/FRA Facilitator Workshop - 1999
- DOT/FRA Track Recurrence & GRMS Training – 1999
- DOT One DOT Commercial Motor Vehicle Safety Workshop – 1999
- DOT/FRA Operating Practices 205 – Railroad Accident Reporting - 1997
- DOT/FRA GE Modern Locomotive Technology - 1997
- DOT/FRA Railroad Accident Reporting - 1997
- DOT/FRA Engineer Certification – 1997
- DOT/FRA Roadbed and Rails -1996
- DOT/FRA Track Workshop - 1996
- DOT/FRA Workforce Diversity and Safety Assurance & Compliance Program – 1996
- DOT/FRA Building an Interest-Based Partnership – 1995
- Dept. of Treasury Federal Law Enforcement Training Center In-service Training – 1995

- Graduate School USDA Organization & Training Needs Assessment – 1995
- Western Management Development Center Management Executive Development Seminar – 1995
- DOT/FRA Dynamics of Transformation & Teambuilding - 1995
- DOT/FRA Creating a Customer-Driven Government Train the Trainer Course – 1994
- DOT/FRA Alcohol and Drug Course - 1993
- DOT/FRA Steam Locomotive Course – 1993
- US OPM EEO Counseling – 1992
- DOT/FRA Bridge Inspection Course – 1992
- DOT/FRA Basic Accident Investigation - 1991
- DOT/FRA Operation Lifesaver Regional Workshop - 1991
- Operation Lifesaver, Inc. Rail-Highway Crossing Safety Training Program – 1988
- DOT Railroad Track Safety Standards - 1986
- Operation Lifesaver, Inc. Instructional Process Workshop - 1985
- DOT Transportation Safety Institute Railroad Accident Investigation Course – 1984
- DOT Transportation Safety Institute Railroad Track & Structures Course - 1983
- University of Wisconsin Dept. of Engineering & Applied Science: Branch Line, Short Line &
- Industrial Railway Track Maintenance and Renovation - 1984
- U.S. Office of Personnel Management - Senior Management Development Course
- White House Workshop
- FLETC Accident Investigation Training
- Department of Agriculture Facilitator and Dispute Resolution Training
- Steven Covey, Seven Habits of Highly Effective People Seminar
- DOT/FRA Steam Locomotive Inspection Training Seminar
- DOT/FRA Advanced Operating Practices Course
- DOT/FRA Advanced Drug and Alcohol Course
- Southern Railway Management Skills Workshop
- DOT/FRA Advanced Railroad Accident Investigation Course
- DOT/FRA Advanced Railroad Track and Structures Training
- EEO Counseling Training

HONORS & AWARDS

• FRA Team Award (Leading H/I Audit)	Jan 2008
• FRA Special Achievement Award for superior achievement	Dec 2007
• FRA Superior Achievement Award	2007; 2000; 1997
• FRA Quality Step Increase	2007; 2006; 2004; 1998
• FRA Special Act Award for Superior Achievement	June 2004
• FRA Quality Step Increase	May 2004
• FRA Special Act Award for Superior Achievement	Oct 2003
• FRA Special Act Award for Superior Achievement in Safety Integration Plan Development and Oversight for Conrail Merger Integration 1998-2001	Feb 2001
• FRA Special Act Award for Superior Achievement	Oct 2000
• US Government Recognition of 20 years of service	FEB 2004
• US Government Recognition of 15 years of service	Feb 1999
• US Government Recognition of 10 years of Service	FEB 1994
• DOT Secretary's Team Award	Nov 1998
• FRA Find the Good and Praise It	June 1998
• FRA Superior Achievement Award	April 1997
• Secretary's Award for Meritorious Achievement	1990
• DOT/FRA Certificate of Appreciation	(1988; 1987; 1985)
• Numerous Performance Cash Bonuses	1984-2008

<u>TESTIMONIES</u>
D. Joe Lydick

Track Technical & Regulatory Compliance Consultant

1. Deposition testimony in the matter of ***Rayfield Jones, Jr. vs. Florida East Coast Railway, LLC***; Case No: 2007-25694-CA-01, In the Circuit Court for the Eleventh Judicial Circuit, In and For Dade County, Florida.

2. Deposition testimony in the matter of ***Robert E. Hunt vs. Union Pacific Railroad Company***; Civil Action No: 172-130, In the District Court for the First Judicial District State of Wyoming, County of Laramie.

3. Deposition testimony in the matter of ***Heritage Square vs. CSX Transportation, Inc. and the City of Atlanta***; Civil Action No: 2997CV144335, In the Superior Court of Fulton County, State of Georgia.

4. Deposition testimony in the matter of ***David M. Powell vs. CSX Transportation, Inc.***; Case No: 16-2008-CA-000783, Division CV-H, In the Circuit Court, Fourth Judicial Circuit, In and For Duval County, Florida.

5. Deposition testimony in the matter of ***Kenneth Davies vs. Florida East Coast Railway, L.L.C.***; Case No: 2008-CA-7271-MA, Division CV-A, In the Circuit Court, Fourth Judicial Circuit, In and For Duval County, Florida.

6. Deposition testimony in the matter of ***Bradley Cauthron vs. Kansas City Southern Railway Company***; Civil Action No: 5:09CV012, In the United States District Court for the Eastern District of Texas, Texarkana Division.

7. Deposition testimony in the matter of ***Robert S. Adix vs. BNSF Railway Company***; Case No: 153-234039-08, In the 153nd Judicial District Court of Tarrant County Texas.

8. Deposition testimony in the matter ***Robert F. Kujon III vs. BNSF Railway Company***; Case No: 141-235887-09, In the 141st Judicial District Court of Tarrant County Texas.

9. Deposition testimony in the matter of ***Harold Basil Walker III vs. CSX Transportation***, ***Inc***.; Civil Action No: CV08-0009BR, In the State Court of Chatham County State of Georgia.

10. Deposition testimony in the matter of ***James N. Kirby vs. Winston Salem Southbound Railway Company***, ***Inc***.; Civil Action No: 09 CVS 1355, In the General Court of Justice Superior Court Division, Guilford County, North Carolina.

11. Trial Testimony in the matter of ***Robert S. Adix vs. BNSF Railway Company***; Case No: 153-234039-08, In the 153nd Judicial District Court of Tarrant County Texas.

12. Deposition Testimony in the matter of ***Leonard Roberts vs. Norfolk Southern Railway Company***; Case No: 24-C-09-004828, In the Circuit Court for Baltimore City.

13. Deposition Testimony in the matter of ***Paul Willis vs. BNSF Railway Company***; Case No: 3422-222212-07, In the 342nd Judicial District Court of Tarrant County Texas.

14. Deposition Testimony in the matter of ***Korey Prather vs. BNSF Railway Company***; Case No: 342-238507-09, In the 342nd Judicial District Court of Tarrant County Texas.

15. Deposition Testimony in the matter of ***Brad Davis vs. BNSF Railway Company***; Case No: 09-CV-282-J, In the United States District Court, District of Wyoming.

16. Deposition Testimony in the matter of ***Benjamin Robertson vs. BNSF Railway Company***; Case No: 09CY-CV03151, In the Circuit Court of Clay County, Missouri.

17. Deposition Testimony in the matter of ***John Bratek vs. BNSF Railway Company***; Case No: 08-1243, In the United States Court for the Central District of Illinois, Peoria Division.

18. Deposition Testimony in the matter of ***James Thornberry vs. CN/Grand Trunk Western Railway and General Motors***; Case No: 2:08-CV13492-MOB-VMM, In the United States District Court for the Eastern District of Michigan.

19. Deposition Testimony in the matter of ***Jeffrey S. Adams vs. Metra***; Case: 07 L 014436, In the Circuit Court of Cook County, Illinois.

20. Trial Testimony in the matter of ***Paul Willis vs. BNSF Railway Company***; Case No: 342-222212-07, In the District Court of Tarrant County, Texas 342nd Judicial District.

21. Supplemental Deposition Testimony in the matter of ***John Bratek vs. BNSF Railway Company***; Case No: 08-1243, In the United States Court for the Central District of Illinois, Peoria Division.

22. Deposition Testimony in the matter of ***Frank Rodriguez vs. CSX Transportation, Inc.***; Case No: 08-62261 CA 04, In the Circuit Court, Eleventh Judicial Circuit, In and for Dade County, Florida.

23. Deposition Testimony in the matter of ***Zachary Chavez vs. Union Pacific Railroad Company***; Case No: 34-2009-00039296, In the Superior Court of the State of California, In and For the County of Sacramento.

24. Deposition Testimony in the matter of ***Bobby Swinson vs. CSX Transportation, Inc. and Ternbux, LLC***; Case No: 10-11130, In the Circuit Court of the Thirteenth Judicial Circuit, In and For Hillsborough County, Georgia.

25. Deposition Testimony in the matter of ***Robert Powell vs. Union Pacific Railroad Company***; Case No: 2:09-CV-01857, In the United States District Court, Eastern District of the State of California.

26. Trial Testimony in the matter of ***Benjamin D. Robertson vs. BNSF Railway Company***; Case No: 09CY-CV03151, In the Circuit Court of Clay County, Missouri.

27. Trial Testimony in the matter of ***David Nielson vs. BNSF Railway Company***; Case No: 08-07-10580, In the Multnomah Circuit Court, Oregon.

28. Deposition Testimony in the matter of ***Matthew Wilbur vs. The Belt Railway Company of Chicago***; Case No: 2008-L-014133, In the Circuit Court of Cook County, Illinois.

29. Deposition Testimony in the matter of ***Roger Brantley vs. Union Pacific Railroad Company***; Case No: 2009-05059, 152nd Judicial District Court of Harris County, Texas.

30. Trial Testimony in the matter of ***Roger Brantley vs. Union Pacific Railroad Company***; Case No: 2009-05059, 152nd Judicial District Court of Harris County, Texas.

31. Deposition Testimony in the matter of ***Charles Boblitz vs. SSP Railroad***; Case No: 24-C-10-006357, In the Circuit Court of Baltimore City, In and For The State of Maryland.

32. Deposition Testimony in the matter of ***Gary Wayne Bailey vs. CSX Transportation, Inc.***; Case No: 3:09-cv-548, In the United States Court for the Eastern District of Tennessee Knoxville.

33. Deposition Testimony in the matter of ***Charles Clemmons vs. BNSF Railway Company***; Case No: 067-239460-09, In the 67th District Court of Tarrant County, Texas.

34. Trial Testimony in the matter of ***Bobby Mathews vs. CSX Transportation, Inc. and Ternbux, LLC***; Case No: 10-11130, In the Thirteenth Judicial Circuit Court, In and For Hillsborough County, Florida.

35. Deposition Testimony in the matter of ***Brent Nuckolls vs. CSX Transportation, Inc***.; Case No: 2009-900111, In the Circuit Court of Talladega County, Alabama.

36. Deposition Testimony in the matter of ***William Harold Campbell vs. CSX Transportation, Inc.***; Case No: 10A23388-5, In the State Court of DeKalb County, State of Georgia.

37. Deposition Testimony in the matter of ***Cassie Dawn Moles vs. Alabama & Tennessee River Railway, LLC***; Case No: CV-2009-900310, In the Circuit Court of Etowah County, Alabama.

38. Deposition Testimony in the matter of ***John Rick Lieber vs. Norfolk Southern Railway Company***; Case No: 09-CI-09025, In the Circuit Court of Jefferson County, Kentucky for the Thirteenth Judicial Court.

39. Trial Testimony in the matter of ***Charles Boblitz vs. SSP Railroad***; Case No: 24-C-10-006357, In the Circuit Court for Baltimore City, In and For the State of Maryland.

40. Deposition Testimony in the matter of ***Jody Lynn Clark vs. Union Pacific Railroad Company vs. Gunderson Rail Services, LLC, D/B/A Greenbrier Rail Services Pine Bluff, D/B/A Gunderson Wheel Services and D/B/A Gunderson, Inc***.; Case No. 5:11-CV-97-SWW, In the United States District Court for the Eastern District of Arkansas Western Division. (1)

41. Deposition Testimony in the matter of ***J. W. DeLoach vs. CSX Transportation, Inc.***; Case No: STCV 1003515, State Court of Chatham County, Georgia.

42. Deposition Testimony in the matter of ***Jody Lynn Clark vs. Union Pacific Railroad Company vs. Gunderson Rail Services, LLC, D/B/A Greenbrier Rail Services Pine Bluff, D/B/A Gunderson Wheel Services and D/B/A Gunderson, Inc***.; Case No: 5:11-CV-97-SWW, In the United States District Court for the Eastern District of Arkansas Western Division. (2)

43. Deposition Testimony in the matter of ***Michael D. Kester vs. BNSF Railway Company***; Case No: DV-10-0920, Montana Thirteenth Judicial District Court, Yellowstone County.

44. Deposition Testimony in the matter of ***Brian Stapert vs. Dakota, Minnesota & Eastern Railroad Corporation***; Case No: 11-CV306F, In the United States District Court for the District of Wyoming.

45. Deposition Testimony in the matter of ***Richard Turney vs. Union Pacific Railroad Company and Chemtrade Refinery Services, Inc***.; Case No: D-0190424, In the 136th Judicial District Court of Jefferson County, Texas.

46. Deposition Testimony in the matter of ***James A. McIsaac vs. CSX Transportation, Inc.***; Case No: 1:11-CV-11455-RGS, In the United States District Court, District of Massachusetts.

47. Deposition Testimony in the matter of ***James Keith Ray vs. The Kansas City Southern Railway Company***; Case No: 2011-0525, In the 14th Judicial District Court for the Parish of Calcasieu, State of Louisiana.

48. Deposition Testimony in the matter of ***Robert Castillo vs. Norfolk Southern Railway Company***; Case No: 24-C-11-003985, In the Circuit Court of Maryland for Baltimore City.

49. Deposition Testimony in the matter of ***David Frasca vs. Norfolk Southern Railway Company***; Case No: 24-C-11-003970, In the Circuit Court of Maryland for Baltimore City.

50. Deposition Testimony in the matter of ***Michael Lydon vs. Norfolk Southern Railway Company***; Case No: 24-C-11-003925, In the Circuit Court of Maryland for Baltimore City.

51. Deposition Testimony in the matter of ***Robert Zapora vs. Norfolk Southern Railway Company***; Case No: 24-C-11-005576, In the Circuit Court of Maryland for Baltimore City.

52. Deposition Testimony in the matter of ***James K. Ray vs. Kansas City Southern Railway Company***; Case No: 2011-000525, 14th Judicial District Court for The Parish of Calcasieu, State of Louisiana.

53. Trial Testimony in the matter of ***Frank Rodriguez vs. CSX Transportation, Inc***.; Case No: 08-62261ca 04, In the Circuit Court of the 11th Judicial Circuit in Miami-Dade County, Florida.

54. Deposition Testimony in the matter of ***Silas Young III vs. Union Pacific Railroad Company***; Case No.: 2009 L 13126, In the Circuit Court of Cook County, Illinois County Department – Law Division.

55. Trial Testimony in the matter of ***John Rick Lieber vs. Norfolk Southern Railway Company***; Case No.: 09-CI-09025, In the Circuit Court of Jefferson County, Kentucky for the Thirteenth Judicial Court.

56. Deposition Testimony in the matter of ***Greg Stanley vs. BNSF Railway Company***; Case No.: 1:11-cv-00178-SA-DAS, In the United States District Court of Northern District of Mississippi.

57. Trial Testimony in the matter of ***Thomas Bolling, Jr. vs. CSX Transportation, Inc***.; Case No.: 2474, Court of Common Pleas, Philadelphia, Pennsylvania.

58. Deposition Testimony in the matter of ***Kenny Hedrick vs. CSX Transportation, Inc.;*** Case No.: 08-C-392W In the Circuit Court of Ohio County, West Virginia.

59. Deposition Testimony in the matter of ***Leland Robinson vs. BNSF Railway Company***; Case No.: 11CV-1852 Division 3, In the District Court of Wyandotte County, Kansas Civil Court Department.

60. Deposition Testimony in the matter of ***Kenneth Beamer vs. CSX Transportation, Inc***.; Case No.: 2:10CV472 In the United States District Court, Southern District of Ohio, Eastern Division.

61. Trial Testimony in the matter of ***Corey Kluver vs. BNSF Railway Company***; Case No.: 27-CV-11-21821 State of Minnesota, County of Hennepin, District Court Fourth Judicial District.

62. Trial Testimony in the matter of ***Robert Powell vs. Union Pacific Railroad Company***; Case No.: 2:09-CV-01857 FDC KJM In the United States District Court Eastern District of California.

63. Deposition Testimony in the matter of ***Thomas Craig vs. BNSF Railway Company***; Case No: 11CV2054 In the District Court of Wyandotte County, Kansas Civil Court Department.

64. Deposition Testimony in the matter of ***Jon W. Scheinost vs. Iowa Interstate Railroad, LTD***; Case No.: 04781 LACV 105941 In the District Court In and For Pottawattamie County, Iowa.

65. Deposition Testimony in the matter of ***Charles Calcote vs. Illinois Central***; Case No.: 251-07-159CIV In the Circuit Court of Hinds County Mississippi First Judicial District.

66. Deposition Testimony in the matter of ***Tarvis Atkins vs. South Central Florida Express***; Case No: 50-2012-CA-004115-MB AA In the Circuit Court of the Fifteenth Judicial Circuit in and for Palm Beach County, Florida.

67. Deposition Testimony in the matter of ***Steve Moreland vs. Norfolk Southern Railway Company***; Case No: 24-C-11-003985 In the Circuit Court of Hamilton County, State of Tennessee at Chattanooga.

68. Trial Testimony in the matter of ***Eugene A. Kinchen vs. CSX Transportation, Inc***.; Case No: 10-57997 CA 04 In the Circuit Court of the 11th Judicial Circuit In and For Miami-Dade County, Florida.

69. Deposition Testimony in the matter of ***Ronald Sweazie vs. Illinois Central Railroad Company***; Case No: CT-004250-06 In the Circuit Court of Shelby County Tennessee for the Thirtieth Judicial District at Memphis.

70. Deposition Testimony in the matter of ***Jeff Cottles vs. Norfolk Southern Railway Company***; Case No: CV-2012-000215, In the Circuit Court of Morgan County, Alabama.

71. Deposition Testimony in the matter of ***Phillip Melvin Barnes vs. Norfolk Southern Railway Company***; Civil Action No: 76158 In the State Court of Bibb County, State of Georgia.

72. Deposition Testimony in the matter of ***Shawn Pollock vs. Union Pacific Railroad Company***; Case No: 12CV2128 JLS JMA, In the United States District Court Southern District of California.

73. Deposition Testimony in the matter of ***David Ciosek vs. Indiana Harbor Belt Railroad Company***; Case No: 2011 L 009189, In the Circuit Court of Cook County, Illinois County Department – Law Division.

74. Deposition Testimony in the matter of ***Kevin Hankes vs. Union Pacific Railroad Company;*** Case No: 09 L 8869 A, In the Circuit Court of Cook County, Illinois County Department – Law Division.

75. Deposition Testimony in the matter of **Salvador *Garibay vs. Union Pacific Railroad Company*;** Case No. 2012 L 1945B, In the Circuit Court of Cook County, Illinois County Department – Law Division.

76. Deposition Testimony in the matter of ***Willie Love vs. Union Pacific Railroad Company***; Case No. B-192,619, In the District of Jefferson County, Texas.

77. Deposition Testimony in the matter of ***Midville River Tract, LLC***; Case 2012-v-0050, In the Superior Court of Burke County, State of Georgia.

78. Trial Testimony in the matter of ***Anthony Cephus vs. The Alabama Great Southern Railroad Company/Norfolk Southern Railway Company***; Case No. CV2013-900383, In the Circuit Court of Jefferson County, Alabama.

79. Deposition Testimony in the matter of ***Thomas O. Morris vs. CSX Transportation, Inc.***; Case No. 1:11-cv-00188, In the United States District Court for the Northern District of West Virginia at Clarksburg.

80. Deposition Testimony in the matter of ***Terry L. Johnson vs. CSX Transportation, Inc.***; Case No. 24-C-13-006318, In the Circuit Court for Baltimore City.

81. Deposition Testimony in the matter of ***BNSF Railway Company vs. Big Country Electric Cooperative, Inc.***; Case No. 13-04-06769, In the District Court of Garza County, Texas 106th Judicial District.

82. Deposition Testimony in the matter of ***Cody L. Krause vs. Union Pacific Railroad Company***; Case No. 128584, In the Iowa District Court for Polk County.

83. Deposition Testimony in the matter of ***Bruce Siemer and Elba Dwayne Case, and Cassie Case vs. CSX Transportation, Inc. and Renessenz, LLC***; Case No. 16-2012-CA-010210, In the Circuit Court, Fourth Judicial Circuit In and For Duval County, Florida.

84. Deposition Testimony in the matter of ***Michael A. Paine vs. CSX Transportation, Inc***.; Case No. CL13-914***, and Gary J. Lascallette vs. CSX Transportation, Inc.***; Case No. CL14-186, Virginia: In the Circuit Court of the City of Suffolk.

85. Deposition Testimony in the matter of ***Stephen Padgett vs. Norfolk Southern Railway Company***; Case No. 2013-RCSC-00722, In the State Court of Richmond County State of Georgia.

86. Deposition Testimony in the matter of ***Bruce Siemer and Elba Dwayne Case, and Cassie Case vs. CSX Transportation, Inc. and Renessenz, LLC***; Case No. 16-2012-CA-010210, In the Circuit Court, Fourth Judicial Circuit In and For Duval County, Florida.

87. Trial Testimony in the matter of ***Shawn Pollock vs. Union Pacific Railroad Company***; Case No. 12CV2128 JLS JMA, In the United States District Court Southern District of California.

88. Deposition Testimony in the matter of ***Jamie Collins vs. CSX Transportation, Inc., et al***; Case No.: 6:13-cv-00206-BFVT, United States District Court, Eastern District of Kentucky, London Division.

89. Deposition Testimony in the matter of ***Robert G. Hays vs. BNSF Railway Company, et al;*** Case No.: 13SG-CC00186-01, In the Circuit Court of St. Francois County State of Missouri.

90. Deposition Testimony in the matter of ***Spencer Abrams vs. CSX Transportation, Inc.***; Case No.: 16-2008-CA-015818, In the Circuit Court, Fourth Judicial Circuit, In and For Duval County, Florida.

91. Trial Testimony in the matter of ***Robert G. Hays vs. BNSF Railway Company, et al;*** Case No.: 13SG-CC00186-01, In the Circuit Court of St. Francois County State of Missouri.

92. Deposition Testimony in the matter of ***David B. Frederick vs. Union Pacific Railroad Company***; Case No.: 6:13-cv-00971-JDL, In the United States District Court for the Eastern District of Texas, Tyler Division.

93. Deposition Testimony in the matter of ***Glenn Stroud vs. CSX Transportation, Inc.***; Case No: 16-2007-CA-002960, In the Circuit Court, Fourth Judicial Circuit, In and For Duval County, Florida.

94. Deposition Testimony in the matter of ***James L. Harrison vs. CSX Transportation, Inc.***; Case No.: 24-C-15-000022, In the Circuit Court for Baltimore City.

95. Deposition Testimony in the matter of ***Jose Ochoa, as Special Administrator of the Estate of Louis Mercado, Deceased vs. METRA***; Case No.: 13 L 10343, In the Circuit Court of Cook County, Illinois County Department, Law Division.

96. Deposition Testimony in the matter of ***Dustin Hampton vs. Union Pacific Railroad Company***; Case No.: 14WY-CV00057-01, In the Circuit Court of the County of Iron, State of Missouri.

97. Deposition Testimony in the matter of ***Kent Bolleheimer vs. CSX Transportation, Inc.***; Case No.: 10-CI-00079, In the Commonwealth of Kentucky, Boone Circuit Court, Division I.

98. Deposition Testimony in the matter of ***Shawn Royal and Regina Royal vs. Missouri & Northern Arkansas Railroad Company, Inc***.; Case No. 4:15-CV-04008-SOH, In the United States District Court Western District of Arkansas, Texarkana Division.

99. Deposition Testimony in the matter of ***Joseph C. Bagatti vs. Bombardier Transportation Services USA Corporation***; Case No.: 2015 CA 001223B, Superior Court for the District of Columbia, Civil Division.

100. Deposition Testimony in the matter of ***Jason DuBois vs. Personnel Staffing, Inc.; Progress Rail Services Corporation; CSX Transportation, Inc., et al***.; Case No.: CV-2012-900349.00, In the Circuit Court Etowah County, Alabama.

101. Deposition Testimony in the matter of ***Jose Ochoa as Special Administrator, of the Luis Mercado, Deceased vs. Northeast Regional Commuter Railroad Corporation, d/b/a METRA***; Case No.: 13 L 010343, In the Circuit Court of Cook County, Illinois County Department, Law Division.

102. Deposition Testimony in the matter of ***Bruce Siemer and Elba Dwayne Case and Cassie Case vs. CSX Transportation, Inc. and RENESSENZ, LLC***; Case No.: 16-2012-CA-010210, In the Circuit Court, Fourth Judicial Circuit, In and For Duval County, Florida.

103. Deposition Testimony in the matter of ***David Delor / John Harof, Sr. vs. The Georgia Department of Transportation / CSX Transportation, Inc. / C & H Paving***; Civil Action File No.: 13CV5165ST, In the State Court of Baldwin County, State of Georgia.

104. Trial Testimony in the matter of ***Bruce Siemer and Elba Dwayne Case and Cassie Case vs. CSX Transportation, Inc. and RENESSENZ, LLC***; Case No.: 16-2012-CA-010210, In the Circuit Court, Fourth Judicial Circuit, In and For Duval County, Florida.

105. Deposition Testimony in the matter of ***James L. Harrison vs. CSX Transportation, Inc.***; Case No.: 24-C-15-000022, In the Circuit Court for Baltimore City.

106. Deposition Testimony in the matter of ***David Ciosek vs. Indiana Harbor Belt Railroad Company***; Case No.: 2011-L-009189, In the Circuit Court of Cook County, County Department – Law Division, Illinois.

107. Deposition Testimony in the matter of ***Donald P. Clark, Jr. vs. Norfolk Southern Railway Company***; Case No: 2014-CP-10-7748, In the Court of Common Pleas In the Ninth Judicial Circuit, County of Charleston, South Carolina.

108. Deposition Testimony in the matter of ***Tony A. Garrett vs. CSX Transportation, Inc.***; Case No.: 1:15-CV-186-HSM-SKL, In the United States District Court Eastern District of Tennessee at Chattanooga.

109. Deposition Testimony in the matter of ***Dioniso Rodriguez vs. CSX Corporation and CSX Transportation, Inc.***; Case No.: 2013-002331-CA-1, In the Circuit Court of the 11th Judicial Circuit, In and For Miami-Dade County, Florida.

110. Deposition Testimony in the matter of ***George Miller vs. BNSF Railway Company***; Case No: 1:15-cv-02268-WJM-NYW, In the United States District Court for the District of Colorado.

111. Deposition Testimony in the matter of ***Johnnie R. Walker vs. Union Pacific Railroad Company***; Case No: 60CV-14-30, In the Circuit Court of Pulaski County, Arkansas.

112. Deposition Testimony in the matter of ***Russell Mathis vs. Union Pacific Railroad Company***; Case No.: 13-CV-00933-DRH-DGW, In the United States District Court, Southern District of Illinois.

113. Deposition Testimony in the matter of ***Damon C. Jenkins vs. Norfolk Southern Railway Company***; Case No.: 01-CV-2016-900107.00, In the Circuit Court of Jefferson County, Alabama.

114. Deposition Testimony in the matter of ***Jason Johnston vs. BNSF Railway Company***; Case no: 15-cv-3685, In the United States District Court, District of Minnesota.

115. Deposition Testimony in the matter of ***Bernard Gillespie vs. BNSF Railway Company;*** Case no: 2014 L 004446, In the Circuit Court of Cook County, Illinois, County Department, Law Division.

116. Deposition Testimony in the matter of ***William S. Nobles vs. CSX Transportation, Inc.***; Case No.: 16-2015-CA-7929; Division: CV-B, In the Circuit Court, Fourth Judicial Circuit, In and For Duval County, FL.

117. Trial Testimony in the matter of ***George Miller vs. BNSF Railway Company***; Case No: 1:15-cv-02268-WJM-NYW, In the United States District Court For the District of Colorado.

118. Deposition Testimony in the matter of ***Norman Smith vs. BNSF Railway Company***; Case No. D-1314-CV-2015-00606, State of New Mexico, County of Valencia, Thirteenth District Judicial Court.

119. Deposition Testimony in the matter of ***Hanover Insurance Company, as Subrogee of Alliance Shippers, Inc. as cosignee of Mars Chocolate North American, LLC vs. Norfolk Southern Railway Company***; Case No. 1:16-cv-04608-TWT, In the United States District Court for the Northern District of Georgia, Atlanta Division.

120. Deposition Testimony in the matter of ***Richard Keen vs. Georgia Southern and Florida Railway Company***; Civil Action File No.2014CV2005, In the Superior Court of Lowndes County, State of Georgia.

121. Deposition Testimony in the matter of ***Austin Thomas vs. Union Pacific Railroad Company***; Case No. 4:16-CV-04052-SOH, In the United States District Court For the Western District of Arkansas, Texarkana Division.

122. Deposition testimony in the matter of ***Wayne S. Nielsen vs. BNSF Railway Company;*** Case No: LALA006551, In the Iowa District Court for Lee County-North.

123. Deposition Testimony in the matter of ***Simon L. Lewis vs. BNSF Railway Company***; Case No: D02CI170001524, In the District Court of Lancaster County, Nebraska.

124. Deposition Testimony in the matter of ***Magdi Anglo vs. National Railroad Passenger Corporation d/b/a AMTRAK***; Case No: 17-013633-CA-01, In the Circuit Court of the 11th Judicial Circuit, In and For Miami-Dade County, Florida.

125. Deposition Testimony in the matter of ***Michael Burgess vs. CSX Transportation, Inc. and TAMKO Building Products, Inc.***; Case No: 10-C-17-001228, In the Circuit Court Frederick County, Maryland.

126. Deposition Testimony in the matter of ***Jeffery s. Buelt vs. BNSF Railway Company***; Case No: CVCV116529, In the District of Iowa, In and For Pottawattamie County.

RAIL Safety Pros, LLC
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Railroad Track Safety Consultant

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FEE SCHEDULE

Professional Services: **January 2018**

- **Advance** Billed against hourly rate $3,000.00

  PAYABLE TO: RAIL Safety Pros, LLC – Tax ID No. 26-4036947

  Once advance is exhausted, services will be billed monthly for work in progress. All additional work, such as site inspections, report updates, testimony, etc., will be billed as work is completed at current rates in effect. Any amount of the advance retainer in excess of the amount billed against it is nonrefundable. The receipt of a Retainer Agreement does not authorize the use of the name of D. Joe Lydick or RAIL Safety Pros, LLC, in any manner in connection with litigation or any other matter. Until the Retainer Agreement is executed and forwarded to RAIL Safety Pros, LLC, the use of the names D. Joe Lydick and/or RAIL Safety Pros is strictly prohibited. Use of the aforementioned names without execution of this Retainer Agreement shall constitute acceptance of the terms of the Retainer Agreement, including payment of retainer fee.

- **Hourly Rate** Billed in .25-hour increments $275.00/hr.
  - Investigation
  - Research
  - Document Review
  - Site Examination
  - Oral/Written Report Preparation
  - Consultation
  - Deposition/Trial Preparation
  - Travel

- **Deposition Testimony** Billed in .25-hour increments $450.00/hr.
  - Rate applies to office/courtroom waiting time as well as actual time testifying.
  - A 3-hour minimum payment of $1,350.00 is required in advance for all depositions.
  - If deposition exceeds the 3-hour minimum period, payment for additional testimony is payable in full prior to continuing testimony.
  - Prepayment of the 3-hour minimum or the total deposition time reserved must be received no later than 72 hours prior to the scheduled deposition date.
  - If the deposition is cancelled less than 72 hours prior to the deposition, only 50 percent of the prepayment amount will be refunded.

- **Trial Testimony** Billed in .25-hour increments $450.00/hr.
  - Rate applies to office/courtroom waiting time as well as actual time testifying.
  - Cancellations of trial testimony within 72 hours of the scheduled testimony will incur a $500 fee.

- **Direct Expenses:**
  - Actual expenses reasonably and necessarily incurred, such as the cost of producing documents and materials, professional support requirements, etc., are additional to the consulting fee and will be billed to the client at cost.
  - Travel expenses, reasonably and necessarily incurred, including first class air fare, lodging, meals, rental car or taxi, parking, and mileage at IRS Standard Rate, will be billed to the client at cost.

- **Terms:**
  - New accounts shall be initiated with an advance payment of $2,500.
  - Services will be billed monthly and payable net 30 days from date of invoice.
  - Accounts over 30 days past due shall accrue interest at the rate of 2% (percent) per month.
  - Payment shall be made to RAIL Safety Pros, LLC.